Structural Designers Handbook

W. F. Scott

STRUCTURAL DESIGNERS' HANDBOOK

GIVING DIAGRAMS AND TABLES FOR THE DESIGN OF BEAMS, GIRDERS AND COLUMNS WITH CALCULATIONS BASED ON THE NEW YORK CITY BUILDING CODE

BY

WILLIAM FRY SCOTT

Structural Engineer, Member American Society for Testing Materials

NEW YORK
THE ENGINEERING NEWS PUBLISHING CO.
1904

PREFACE.

This handbook, essentially a diagrammatic treatise on the subject of Structural Design, contains also a full tabulation of the properties of market shapes of materials.

It is presented to the architectural and engineering professions with the thought that it may be the means of shortening and possibly eliminating much of the computation and drudgery which are necessary accompaniments of structural designing.

It is hoped that it may prove useful to the expert designer, since the diagrams presented are time-saving devices; useful and suggestive to the non-expert and the student, since the diagrams illustrate graphically the relations of the various factors of proportion, span loading, etc., for the variable conditions of ordinary practice.

Throughout the work the New York Building Code has been followed, because it is everywhere recognized as conservative and safe.

W. F. S.

New York, July, 1904.

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STRUCTURAL DESIGNERS' HANDBOOK.

Part I. Synopsis of Mechanics of the Beam and Column.

CHAPTER I.—BEAMS.

The resistance of a beam to bending is the main item that determines its strength, so much so, in fact, that the other determinative factors are often forgotten or at least not given due attention. The resistance to bending depends upon the span of the beam, its cross section, the unit stress permissible in the material, and the manner of support of the beam.

The **SPAN** of a beam where the latter is simply supported, is measured between centers of members by which the beam is supported in case of end framing connections, or, in case the beam rests upon a wall or other support, between centers of bearing plates.

The **CROSS SECTION** of the beam affects its strength in a manner depending upon the moment of inertia and the depth of the cross section, both measured from the neutral axis. For any given cross section these quantities are constant, and hence do not enter into the diagrams presented in this book since each standard shape is represented by a separate diagram, or, in the case of spandrel or grillage beams, each shape is represented by a line on the diagram.

For cases where calculations of beams are made without using diagrams, the tables given in Chapter VIII give the **SECTION-MOMENT** (symbol M_s) for each structural steel shape. The section-moment* is a quantity obtained by multiplying the maximum *allowable* fiber stress by the moment of inertia of the cross section, and dividing by the distance of the extreme fiber from the neutral axis. It will be recognized to be the resisting property that opposes the bending moment (symbol M_b) of the external forces.

*The author has taken the liberty to coin this word, because of the confusion of terms in reference to this property of a beam. Also, because there is as much distinction between the section-moment and the external bending moment as between the fiber stress and the external load producing it.

By computing the maximum bending moment due to the external forces, and referring to the values of section-moments, given in the aforementioned tables, the proper structural shape to be used can readily be found. (It should be noted that the section-moments are given in foot-pounds for angles and tees but in foot-tons for all other shapes.)

The maximum **UNIT STRESS** permissible in structural steel beams is throughout this book taken at 16,000 lbs. per sq. in. This is in accordance with the provisions of the "Code" (N. Y. C.), with the tables contained in the pocket books of the steel manufacturers, and with nearly all specifications for beamwork in buildings.

The **MANNER OF SUPPORT** of the beam affects its strength quite materially. By far the most common case is where the ends are "simply supported." The diagrams, with the single exception of that for grillage beams,* are based exclusively upon this case, and for convenience in calculating the strength of beams under other conditions of support, the following formulas are given:

$WL = M_B$, for a *cantilever beam* with load concentrated at the end. (1)
" $= 2\ M_B$, for a *cantilever beam* with load uniformly distributed. (2)
" $= 4\ M_B$, for a *simple beam* supported at the ends, and load concentrated in the middle. (3)
" $= 8\ M_B$, for a *simple beam* supported at the ends, and load uniformly distributed. (4)
" $= 5.2\ M_B$, for a *beam with one end fixed* and the other end supported, and load concentrated near the middle. (5)
" $= 8\ M_B$, for a *beam with both ends fixed,* and load concentrated in the middle. (6)
" $= 12\ M_B$, for a *beam with both ends fixed,* and load uniformly distributed. (7)

where
W. = Total load on beam, in tons or pounds.
L. = Span of beam, in feet.
M_B. = Section-moment, in foot-tons or foot-pounds.

The question of the **DEFLECTION** of beams is one of special importance in many cases, as for instance, where plastered ceilings are involved. One four-hundredth of the span should generally be the limit for such cases. The diagrams for spandrel and floor beams are calculated for a limiting deflection of this amount. For general practice there should be no objection to this limit,

*These are virtually cases of cantilever beams.

even where plastered ceilings are not involved. For bridge work and some other cases, the deflection of beams is not usually considered, in which case the diagrams may be used by placing a straight edge on the left hand portion of the curve and determining the intersection of this portion produced for the span and spacing required.

For convenience in calculating the strength of beams for a limiting deflection of one four-hundredth of the span, under the several more common conditions of support, the following formulas are given:

$W L^2 = 0.566\ h\ M_s$, for a *cantilever beam* with load concentrated at the end. (8)

" $= 1.51\ h\ M_s$, for a *cantilever beam* with load uniformly distributed. (9)

" $= 9.05\ h\ M_s$, for a *simple beam* supported at the ends, and load concentrated in the middle. (10)

" $= 14.5\ h\ M_s$, for a *simple beam* supported at the ends, and load uniformly distributed. (11)

" $= 10.35\ h\ M_s$, for a *beam with one end fixed*, and the other end supported, and load concentrated near the middle. (12)

" $= 35.2\ h\ M_s$, for a *beam with both ends fixed*, and load concentrated in the middle. (13)

" $= 72.5\ h\ M_s$, for a *beam with both ends fixed*, and load uniformly distributed. (14)

where

W = Total load on beam, in tons.

L = Span of beam, in feet.

h = Depth of beam, in inches (for symmetrical sections only).

M_s = Section-moment, in foot-tons.

The upward reaction of the support against the beam, is an important matter as affecting its strength, and it should be kept in mind in every case of their design.

THE END REACTIONS due to uniformly loading standard beams up to their full allowable flexure strength for various spans are given on Diagram No. 28 in Chapter VI. These are the external loads which are resisted by stresses in the webs of the beams. These stresses are of importance principally in very short beams loaded to their full bending capacity. The following is a discussion on these stresses, all mathematical reasoning being omitted for the sake of brevity:

The end reaction, divided by the area of the web (usually considering the height of the beam times the web thickness), equals the unit intensity

of the vertical shearing stress in the web. While not absolutely correct, the above statement is practically true. This unit intensity of vertical shear is equal to the unit intensity of shearing stress acting at angles of 45° with the neutral axis of the beam, and these again are equivalent to direct compressive and tensile stresses in the web, acting at right angles to each other. The compressive stress will evidently tend to buckle the web, while the tensile stress will tend to hold it in its true plane. Neglecting the value of the tensile stress (which is really indeterminate), or in other words, placing the "factor of ignorance" on the safe side of the problem, then the unit intensity of the compressive stress should for safety not exceed the *allowable* unit compressive stress in any strip of the web between the fillets, at 45° with the neutral axis of the beam.

When the ratio of the length of any such strip to the least radius of gyration of the web exceeds 110 (which is the point at which the shearing strength practically becomes equal to the compressive strength) then the allowable end reaction should be determined by the permissible *compressive* stress in the web; when the ratio is less than 110 the allowable reaction should be determined by the permissible *shearing* stress.

Tables opposite each of the Diagrams Nos. 1 to 15 and also the tables for I-beams and channels in Chapter VIII give these values, according to the "Code" (N. Y. C.) requirements for shearing and compressive stresses. (See Chap. XI on unit stresses.)

BUCKLING OF COMPRESSION FLANGE.—Floor beams usually have their compression flanges supported laterally by the floor arches, and the girders have their flanges supported by the beams which frame into them. When the lateral supports are so far apart that the upper flange of the beam, considered as a column, would begin to deflect under a load equivalent to the compressive stress in that flange, then the load on the beam in flexure should be reduced. In the Diagrams Nos. 1 to 15, this reduction of load is given for different lengths of flange between lateral supports. As is explained in Chapter III, a scale at the bottom of these diagrams gives for any unsupported length of flange in feet the maximum percentage of full load that can be safely carried without danger of buckling of the upper flange.

LOADS ON BEAMS.—The bending stress due to a *concentrated* load depends upon the amount of the load and its position on the beam with reference to the span. The most unfavorable position is at the center, the bending stress in this case being twice that due to the same load *uniformly distributed.* As the load moves towards either end, the bending stress produced by it decreases. In each case, the greatest stress in the beam is directly below the load.

Thus, when the load is not at the center of the beam, the stress at the center due to the load is less than the stress directly under the load.

This is of importance in the case of combined loads. The bending stress due to a combination of concentrated loads (with or without a uniform load) is greatest at a point of the beam *near the center of gravity of the loads.* Moreover, the maximum stress is somewhat less than the sum of the maximum stresses due to each of the uniform or concentrated loads acting alone.

CONVENTIONAL METHODS OF CONSIDERING LOADS ON BEAMS.

The scope of the present book will not admit of discussing more than the two special cases, under this heading, that are involved in the methods of constructing the diagrams on beamwork. The first is a consideration of the conventional treatment of uniform loading on floor girders in terms of unit floor area. The second is a discussion of the two principal conventional methods of considering the loads on grillage beams.

CONVENTIONAL METHOD OF TREATING LOADS ON FLOOR GIRDERS.—The diagrams in Chapter III on beams refer directly to uniform loading per square foot of floor. Since the main girders in a floor support the floor beams, and thus carry what are essentially concentrated loads, it may at first sight be thought that the same diagrams could not be directly applicable to girders. This is not the case, however, as will be evident by considering a girder with a single concentrated load in the center. The strength of this girder is ordinarily found in terms of the load and the span of the girder. The external bending moment, for instance, is equal to one-fourth of the span in feet multiplied by the amount of the concentrated load in tons or pounds, according to the units adopted by the designer. Suppose, now, that this concentrated load is due to two beams framing into the girder, one on each side, and in order to simplify the problem, suppose the spans of these two beams to be equal, of the value B.* Then if L equals the span of the girder, W the load-

*If the beams framing into the girder have spans that are unequal, then (B) is equal to one-half the sum of the spans.

ing on the beams in pounds per square foot of floor, M_b the external bending moment on the girder, and C the concentrated load on the girder due to the two beams, we have,

$$M_b = \frac{L\ C}{4}, \text{ and } C = \frac{W\ B\ L}{2},$$

or

$$M_b = \frac{W\ B\ L^2}{8}.$$

But if the girder were considered loaded with a uniform load W per square foot, distributed over the area represented by breadth B and length L, the moment on the girder would be

$$M_b = \frac{W\ B\ L^2}{8},$$

which is the same as found by the other method.

It will be found on analysis, that the result obtained in this particular case can be safely applied to the case of a girder with any number of beams framing into it. Thus, so long as girders are designed in terms of the actual area enclosed within the parallelogram having a breadth B and length L, and the uniform load W per unit area, the diagrams will be found to give safe results. In all cases where odd numbers of beams (even number of spaces between ends) frame into girders, the values given in the diagrams are just as true for girders as for simple beams, provided the sum of the spans of the floor arches does not exceed length L, while in the case of even numbers of beams framing into girders, the diagrams give values that are a little safer than in the other cases. There are only three cases where the difference is perceptible—where two, four or six beams frame into the girder (respectively three, five or seven spaces between the ends of the girder). They differ from the diagram values as follows:

Two beams,	$M_b = \frac{1}{9}\ W\ B\ L^2 = 0.111\ W\ B\ L^2$
Four beams,	$M_b = \frac{6}{50}\ W\ B\ L^2 = 0.120\ W\ B\ L^2$
Six beams,	$M_b = \frac{6}{49}\ W\ B\ L^2 = 0.122\ W\ B\ L^2$
Uniform load,	$M_b = \frac{1}{8}\ W\ B\ L^2 = 0.125\ W\ B\ L^2$

For more than six beams, the moment practically equals that for uniform load. For the three cases given above, the permissible loading on the girder is respectively 12½%, 4% and 2% greater

than that given by the diagrams. Therefore in these three cases the results obtained from the diagrams can be corrected as follows:

Increase the load per square foot by $12\frac{1}{2}\%$, 4% and 2% respectively, for the three cases; or increase the spacing of the girders by the same percentages, in which case the load is not changed; or increase the span of the girders 6%, 2% and 1% respectively.

CONVENTIONAL METHODS OF CONSIDERING LOADS ON GRILLAGE BEAMS.—Correctly speaking there is only one condition that actually represents the effect of the load on a grillage footing, but usage in design is sometimes contradictory when the premises are not well understood. This is well illustrated in two methods of designing grillage beams in footings. For instance, one method considers the active portion of the load to be on the projecting length of the beam only, and the bending moment calculated at the edge of the base above, or the edge of the tier of beams, if such exists, is assumed to be the maximum bending moment on the footing. The second method considers the bending moment at the center, due to all the external forces acting on the footing.

The formulas for the two methods are:

First Method:

$$M_b = \frac{W a^2}{2} \qquad (15)$$

where M_b = Bending moment under edge of tier above,
W = Load per unit length of beam,
a = Length of projecting portion of beam.

Second Method:

$$M_b = W \frac{a}{2}\left[a + \frac{b}{2}\right] = \frac{W a^2}{2}\left[1 + \frac{b}{2a}\right] \qquad (16)$$

where M_b = Maximum bending moment at center,
b = Width of base or tier above,
a = Projecting portion of beam.

Comparing these two formulas, the maximum bending moment on the footing is $1 + \frac{b}{2a}$ times the bending moment occurring under the edge of the tier above.

It will thus be seen that the bending moment at the center is greater by the percentage $\frac{b}{2a}$ than the moment at the edge of the base. The use of formula (15) would therefore appear to lead to

an unsafe design. It has been found by experience, however, that the concrete in the footing goes to form a composite beam of considerable excess strength over that of the steel beams alone, and designs have been made by experienced engineers in which this excess has apparently been assumed to be from 25% to 75% (on the basis of 16,000 lbs. per sq. in. maximum fiber stress).

This question will be further discussed in the Chapter on Grillage Beams.

CHAPTER II.—COLUMNS.

A column is designed to resist forces which usually act in the direction of its axis—the center of gravity of these forces may coincide with or may be a short distance from and parallel to this axis. In the former case they are called "Concentric" loads, in the latter case "Eccentric" loads.

CONCENTRIC LOADS.—A column in direct compression, if secured against lateral deflection might safely be designed to develop the full allowable unit compressive stress for the material. However, if any slight deflection of the axis from a straight line takes place, the load acts to produce flexure stresses in addition to the direct compressive stress. For this reason columns cannot safely be designed for the full working unit stress that is allowed for the material in simple compression. This tendency of a column to deflect depends upon the ratio between its length and least radius of gyration. This ratio may be called the *ratio of slenderness* of the column. Column formulas based on this ratio exist in great variety —the "load" diagrams for columns (in Part III) are based on the formulas prescribed by the "Code" (N. Y. C.).

ECCENTRIC LOADS.—When a column is eccentrically loaded the stresses produced in any cross section are a uniform compressive stress of the same value as would be produced by a concentric load of the same amount, and the stresses due to a bending moment caused by the eccentricity of the load. The sum of these stresses is a maximum on the compressive side of a cross section at or near the bottom of the bracket which transmits the larger share of the eccentric load into the column, and it is a minimum at the foot of the column. However, as the cross section of a column is usually made uniform throughout its length, it is not necessary to

consider any but the maximum stresses. This cross section is therefore designed so that the sum of the compressive stresses due to the bending moment and the compressive stress due to a concentric load of the same amount shall not exceed the safe unit compressive stress allowable on a column with the given ratio of slenderness.

The area of the cross section of a column required to resist the compressive stress due to a concentric load is found by dividing the load by the allowable unit stress obtained from the aforementioned column formulas, while the area required to resist the compressive stresses due to the eccentric load is a much more complex problem. It is usually based on the following considerations: Fig. 1 shows a column eccentrically loaded. The load P_e is applied at a distance *a* from the neutral axis of the column and the extreme fiber of the column is at a distance *y* from its neutral axis. Then the bending moment produced in the column by the eccentricity of loading is expressed by the formula.

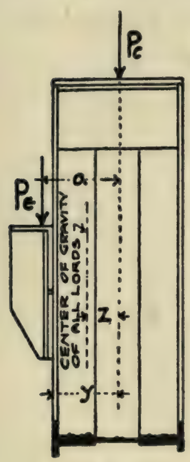


Fig. 1.

$$M_b = z\,P \qquad (17)$$

where P = Pe + Pc and z equals distance from the neutral axis to the center of gravity of the loads; and the required section-moment is

$$M_s = \frac{S\,I}{y} = \frac{S\,A\,r^2}{y}, \qquad (18)$$

in which S represents the allowable unit stress in the material, I the moment of inertia of the cross section (I being equal to the area A of the section, times the square of *r*, the radius of gyration).

Now putting the section-moment equal to the external bending moment,

$$z\,P = \frac{S\,A\,r^2}{y},$$

from which

$$A = \frac{P}{S} \quad \frac{z\,y}{r^2} \qquad (19)$$

This expression shows that the area required for bending alone is given by considering the eccentric load as a pure concentric load, and multiplying the area $\frac{P}{S}$ thus considered, by a factor $\frac{z\,y}{r^2}$ which depends upon the distance of the center of gravity of the

loads from the neutral axis, the distance of the extreme fiber from the neutral axis, and the radius of gyration of the section. *The term z y is conveniently called the coefficient of eccentricity.*

The factor $\frac{z\,y}{r^2}$, it should be noted, simply gives the percentage of area of a column section as calculated for a concentric load which it is necessary to add to take care of the added stress due to the eccentricity of the load. If the load is eccentric on both axes, then two of these percentages must be determined. Thus the area of a column which is eccentrically loaded in both principal directions is expressed by the formula:

$$A = \frac{P}{S}\left[1 + \frac{z'\,y'}{r'^2} + \frac{z''\,y''}{r''^2}\right]$$

Where $P = P_e + P_c$ = total load on the column.

S = allowable unit stress in the material and determined by the minimum radius of gyration.

Z', y' and r' being taken about the minor axis and z'', y'' and r'' about the major axis.

This subject will be continued in Chapter VII on steel columns.

Part II.—Beamwork.

CHAPTER III.—FLOOR FRAMING.

UTILITY OF DIAGRAMS.—The strength of beams and girders* is given in almost all books on the subject in terms of the total uniform load which they will safely carry. If the beams are used in floors, where the loads are mostly uniform and known in terms of the unit floor area, it is necessary to perform a more or less lengthy calculation to determine the spacing of the floor beams under consideration, and these calculations have to be repeated for every problem. The diagrams in this chapter eliminate the need for calculations of this sort, and enable the proper spacing of beams and girders of an assumed size to be read off at once for any given span, and any given load per square foot of floor.

An equally important application of the diagrams may be made at an earlier stage of the work. It is quite generally recognized that the time to consider economy of material in the framework of a building is at that stage of the development of the plan when the architect and the owner are "getting together" on the matter of what can be done for the money to be invested. Any excess in the span of beams and girders over the minimum required by the particular conditions governing in each case, adds to the cost of the steelwork. The diagrams in this chapter will be found useful in deciding upon an economical arrangement of columns in such cases. It will be evident that when once the spacing of columns is fixed, there can be comparatively little room for consideration of economy in beam arrangement.

Thus these diagrams will be found valuable in preliminary work for two special reasons: (1) Several possible arrangements of beams and girders can be studied in a few minutes by their use. (2) The weight of steel for every case being given, the most economical

*The word "beam" is used throughout this book to signify beams used as joists or rafters, and the word "girder" to signify beams used as supports for joists or rafters.

arrangement of beams and girders will be evident without any figuring.

The first fifteen diagrams cover the standard weight of I-beam in each of the various sizes, and their use is extended to channels and special I-beams by means of the supplementary tables which are placed opposite each diagram. The next six diagrams cover angles and tees used as beams. A special diagram is added to this group, which extends its use to the consideration of minor concentrated loads by giving an equivalent uniform load.

DIAGRAMS FOR I-BEAMS AND CHANNELS (NOS. 1 TO 15):— A separate diagram has been drawn for each standard I-beam; thus Diagram No. 6 refers to an 8-in. beam. Each diagram gives span, spacing, and load as three variables, any two of which determine the third for that particular section of beam. The abscissas represent spans, and the ordinates, the spacings (distance apart) of beams. Diagonal lines on the diagrams represent loads per square foot of floor.

It will be seen on inspecting the diagrams that the right hand portion of the "load" lines makes a smaller angle with the vertical than the left hand portion of the lines. This is due to the fact that above a certain length of span the deflection becomes the limiting factor in determining load or spacing. To the left of the bend point the maximum fiber stress is the limiting factor. Thus the diagrams always insure that the deflection will not exceed one four-hundredth of the span. In case it is desired to neglect the question of deflection, the left hand portion of the lines may evidently be prolonged, for instance, by means of a straight edge to the desired span.

The method of using the diagrams is quite simple. Suppose in a given floor plan it is desired to use a certain size of beam. On the diagram for this size of beam take an abscissa equal to the span and follow up to the diagonal line representing the loading per square foot of floor. The horizontal line indicates the proper spacing of the beams.

On the right hand edge of each diagram will be found a *scale of weights* in pounds. This scale represents the weight of the beams per square foot of floor. Thus, having found the proper spacing as above, the same horizontal line is followed to the right, and on the scale of weights will be found the corresponding equivalent weight of the beams per square foot of floor.

One other scale will be found on the diagrams, at the lower edge, just above the scale of spans. Here is given for dif-

ferent abscissas a series of percentages. They represent the maximum percentage of full "bending" load that is allowable *consistent with safety against buckling of the upper flange of the beam.* The abscissas to be used in referring to this scale are the *unsupported* length of flange, not the *span* of the beam. The use of this scale will be clear from what has been said in Chapter I on the subject of "Buckling of Compression Flange."

Then, to summarize, each of the diagrams gives:*

(a) The allowable uniformly distributed live and dead load on floor beams and girders, in pounds per square foot of floor, for any span or spacing in feet.

(b) The allowable spacing center to center of floor beams or girders in feet or fractions thereof, for any span and any uniform loading in pounds per square foot of floor.

(c) The allowable span in feet for floor beams or girders, for any uniform loading and any spacing.

(d) The weight of steel in any of these floor beams or girders in pounds per square foot of floor.

(e) The percentage of load allowable on a single beam or girder for any unsupported length of top flange in feet.

SUPPLEMENTARY TABLES (NOS. 1 TO 15):—In connection with each diagram, just referred to, is given a table which greatly extends its use. The diagram, it will be remembered, gives values for the "standard" weight of I-beam. The table facing the diagram gives a set of factors for the "special" weights of I-beam of the same depth, and for all the "standard" and "special" weights of channel of the same depth; also, for zees and bulb angles of corresponding depth. By the use of these factors, which are expressed in per cent. of the diagram values, the results obtained from the diagram are directly applicable to all the other weights and sections given in the supplementary table.

For instance, when a value of load or spacing has been found from the diagram for a 5-in. I-beam, weighing 9.75 lbs. per ft., and it is desired to use instead a 5-in channel, weighing 6.5 lbs. per ft., the percentage factor 63 is read from the supplementary table, and the load or spacing found, when multiplied by 0.63 gives the correct load or spacing for the channel.

A further column in the tables gives the maximum *end reactions* allowable to avoid buckling or shearing in the web of the beam over the supports. For short beams it is always essential to see that the load permitted on the beam for bending does not give a greater end

*On a basis of a maximum stress of 16,000 lbs. per sq. in.

reaction* than that shown in the table. It the actual reaction exceeds the tabular value, then the load should be reduced, or another beam should be used, or stiffener angles should be riveted to the web.

It will be observed that the supplementary tables give parallel sets of values, headed respectively, "Carnegie, Cambria, Jones & Laughlins, Phoenix;" "Pencoyd" and "Passaic."

The reason for this is the following: The standard sections of steel beams, channels and angles adopted by the American Association of Steel Manufacturers in 1896 are now adopted by nearly all the rolling mills in this country. There are some exceptions, however, that are typical; therefore the tables have been compiled with a view of making this hand-book a compendium of the market shapes used in structural work. The values for percentages and end reactions were, however, computed by the author. No attempt is made to give a list of rolling mills. Neither was there an inclination to discriminate in favor of the mills mentioned in the classification. The principle adopted in making up these tables, was to take the Manufacturer's "Pocket Books" most generally found in offices as a basis for the classification. Only distinctive and well marked differences have been taken into account.

Summarizing now the values which may be obtained from the supplementary tables. Each table gives:

(f) The percentage of uniform load in pounds per square foot allowable on all market shapes of the same depth of I-beams, channels, deck-beams, and zees, other than the standard weight section represented by the diagram.

Example.—Suppose for a girder of 20-ft. span and 14-ft. spacing, it is desired to use a 15-in. I-beam, the load being assumed at 200 lbs. per sq. ft. of floor. A 15-in., 60-lb. beam will carry 151 lbs. and the table opposite the diagram for this beam gives 130.7% of this value for a 15-in. 80-lb. beam, which is equivalent to 199 lbs. for the allowable load.

(g) The same percentage factor may be taken to express the percentage of the diagram values for spacing of beams or girders, if it is more convenient to use the diagram values for the loading.

Example:—Suppose for a girder of 20-ft. span and an assumed load of 150 lbs. per sq. ft. of floor, it is desirable to use 15-in. beams, spaced 18½ ft. c. to c. The spacing of 15-in. 60-lbs. beams for the foregoing can only be 14½ ft., and the table opposite gives 130.7% for the 15-in. 80-lb., which is equivalent to 18¾ ft. c. to c.

(h) The allowable end reaction for safety of the web without reinforcement for buckling. (See also the diagrams for end reactions given in Chapter VI.)

DIAGRAMS FOR ANGLES AND TEES.—The diagrams on angles and tees, Nos. 16 to 21, will be understood from the preceding

*The end reactions resulting from any uniform load can be obtained directly from Diagram No. 28 without any arithmetical work.

description of Diagrams Nos. 1 to 15, without any difficulty. They give the relation between span, spacing and load in exactly the same way as the diagrams for I-beams. A table accompanies each of the diagrams which gives the percentage factors for shapes of the same depth but of different weights and width of horizontal flange.

DIAGRAM FOR REDUCING THE VALUE OF A CONCENTRATED LOAD TO AN EQUIVALENT VALUE OF UNIFORM LOAD PER UNIT AREA.—The Diagram No. 22 shows for any area of floor tributary to a beam, the uniform load per square foot which is equivalent to a concentrated load of any given amount in the middle or at any other point on the beam.

The abscissas, in Diagram No. 22, represent floor areas in square feet; the ordinates represent equivalent uniform load per square foot of floor in pounds. The diagonal lines represent concentrated loads in pounds. The scale of ordinates changes for a change in the position of the load along the beam. The position of the load is expressed as a fraction of the span, and ordinate scales for different values of this fraction are given to the right of the diagram. In any particular problem, the proper scale of ordinates is to be selected from these. The ordinate scale at the left, to which the main diagram is drawn, is for a load at the middle of the span.

To use the diagram take an abscissa equal to the floor area tributary to the beam, and follow up to the diagonal line representing the concentrated load. Select the proper ordinate scale for the position of the load and read the equivalent uniform load. This is to be added to the normal uniform load on the floor, and this sum used as load in referring to Diagrams Nos. 1 to 21. In case the position of the load is not represented exactly by any ordinate scale, use the nearest one or interpolate.

Example.—Suppose a post from a stair platform carries 10,000 lbs. to a point 4 ft. from the end of a 20-ft. girder; suppose one-half the sum of the spans of the beams framing into the girder is 20 ft., thus giving 400 ft. tributary floor area upon which the normal live and dead load is 160 lbs. per sq. ft.

The diagram gives 32 lbs. per sq. ft. for a load concentrated $^2/_{10}$ of the span from the end. Thus it is necessary to design this girder for a uniform load of 192 lbs. per sq. ft. A 20-in. 65-lbs. I-beam was strong enough for the normal loading, while the added concentrated load calls for a 20-in., 80-lbs. I-beam.

The end reaction should be taken as half the sum of the nor-

mal uniform load and the equivalent uniform load (equal to ½ total uniform load times floor area carried). The result so obtained is a trifle too high, but the error is on the safe side.

TABLES OF PROPERTIES OF SHAPES.—It will be proper here to again refer to the series of tables in Chapter VIII, which give the properties, including the section-moment for all the standard and special structural shapes of steel. These tables can be used for a variety of purposes. The section-moments will be particularly useful for beam design. Thus, when an I-beam has been specified and it is desired to use a channel in its stead, the section-moments given in these tables, can be used to find one that has an equivalent strength to the one specified. For instance, if a 9-in. 21-lb. beam was called for, then from the tables it will be seen that a 12-in. 20½-lb. channel will carry the same load, or if it is to carry only half the load, a 9-in. 13-lb. channel will be satisfactory.

Diagrams 1 to 15

For I-Beams and Channels, with Supplementary Tables

Diagrams 16 to 21

For Angles and Tees

Diagram 22

For Reducing the Value of a Concentrated Load to an Equivalent Value of Uniform Load

THE DIAGRAM ON OPPOSITE PAGE GIVES:

(*a*) The allowable *uniform load* on **3-in. x 5.5 lb. I-beams** in lbs. per sq. ft. of floor.

(*b*) The allowable *spacing* C. to C. in ft. for any span and any uniform load.

(*c*) The allowable *span* in ft. for any uniform load and any spacing.

(*d*) The weight of steel in lbs. per sq. ft. of floor.

(*e*) The percentage of load allowable for any unsupported length of top flange in feet.

THE TABLE FOLLOWING GIVES:

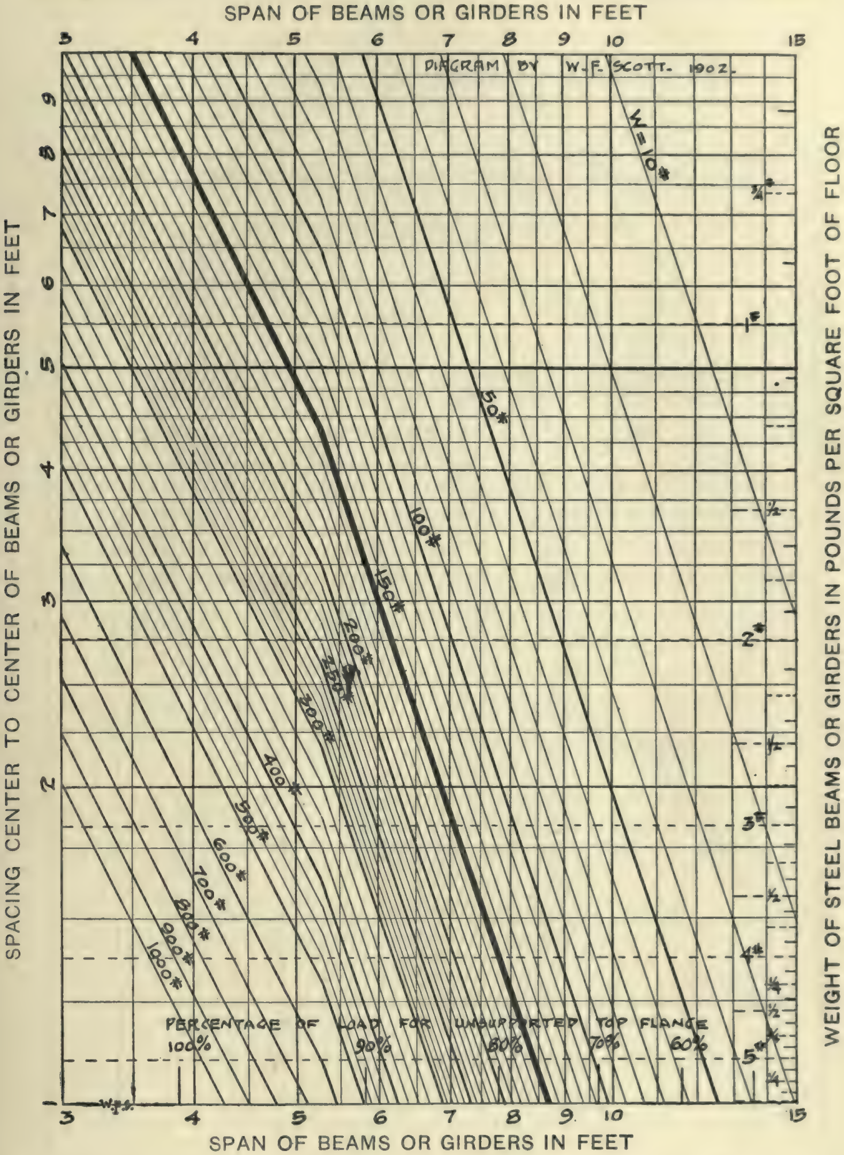
(*f*) The percentage of load allowable on special shapes other than the above standard.

(*g*) The same percentage factor to be used for spacing instead of load.

(*h*) The allowable end reaction for safety of web without reenforcement for buckling.

3″ I Beams	Carnegie, Cambria, Jones & Laughlins Phœnix			Pencoyd			Passaic		
Weight per foot.	Web thickness.	Allowable load per square foot.	Allowable end reaction.	Web thickness.	Allowable load per square foot.	Allowable end reaction.	Web thickness.	Allowable load per square foot.	Allowable end reaction.
Lbs.	Ins.	% W.	Tons	Ins.	% W.	Tons	Ins.	% W.	Tons
5.5	0.17	100	2.3	0.17	97	2.3			
6.5	0.26	108	3.5	0.24	106	3.2			
7.5	0.36	116	4.8	0.34	115	4.6			
3″	CHANNELS								
4	0.17	65	2.3	0.17	65	2.3			
5	0.26	71	3.5	0.25	71	3.4			
6	0.36	82	4.8	0.35	82	4.7			
3″	ZEES								
6.7	0.25	113							
8.4	0.31	140							
9.7	0.37	151							

Diagram No. 1 **3-in. x 5.5-lb. I-Beams**



Safe loads given include weight of beam and maximum fiber stress, 16,000 lbs. per sq. in. maximum deflection 1-400th of the span.

THE DIAGRAM ON OPPOSITE PAGE GIVES:

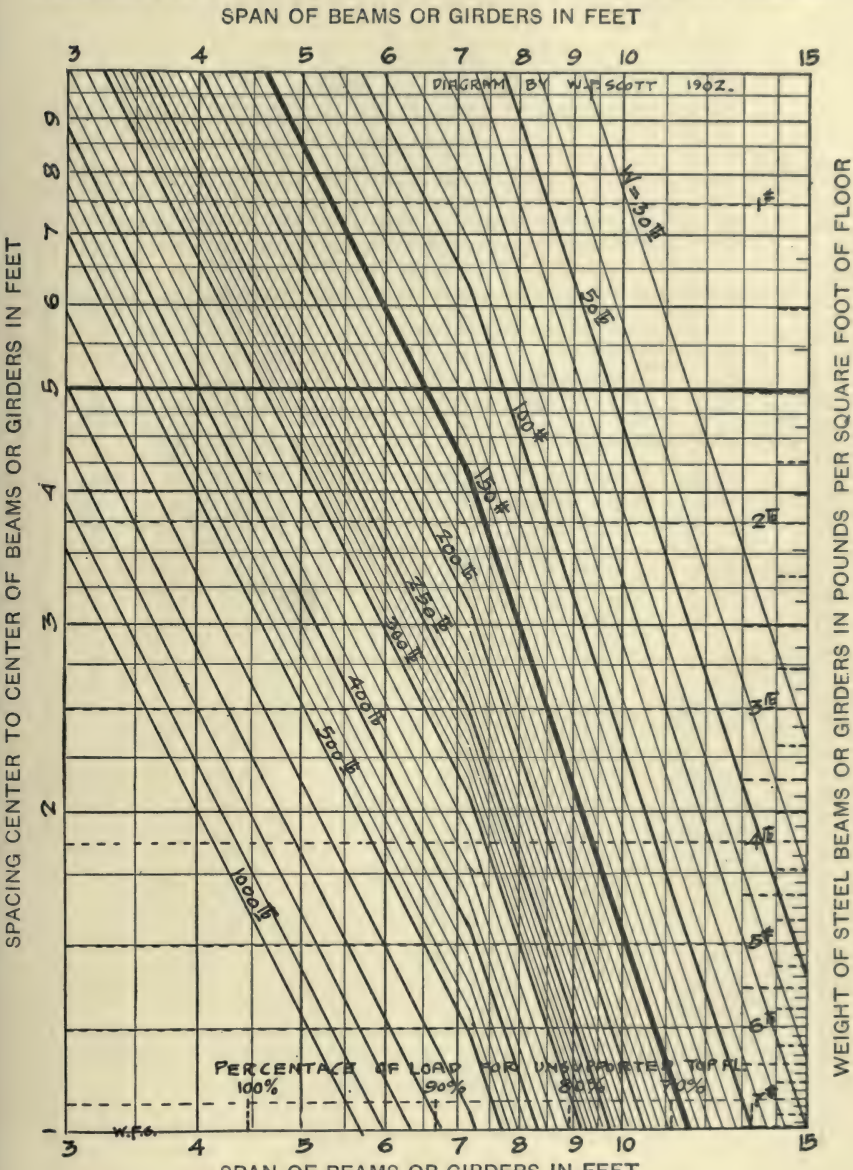
(*a*) The allowable *uniform load* on **4-in. x 7.5 lb. I-beams** in lbs. per sq. ft. of floor.
(*b*) The allowable *spacing* C. to C. in ft. for any span and any uniform load.
(*c*) The allowable *span* in ft. for any uniform load and any spacing.
(*d*) The weight of steel in lbs. per sq. ft. of floor.
(*e*) The percentage of load allowable for any unsupported length of top flange in feet.

THE TABLE FOLLOWING GIVES:

(*f*) The percentage of load allowable on special shapes other than the above standard.
(*g*) The same percentage factor to be used for spacing instead of load.
(*h*) The allowable end reaction for safety of web without reenforcement for buckling.

4″ I Beams	Carnegie, Cambria, Jones & Laughlins, Phœnix			Pencoyd			Passaic		
Weight per foot.	Web thickness.	Allowable load per square foot.	Allowable end reaction.	Web thickness.	Allowable load per square foot.	Allowable end reaction.	Web thickness.	Allowable load per square foot.	Allowable end reaction.
Lbs.	Ins.	% W.	Tons.	Ins.	% W.	Tons.	Ins.	% W.	Tons
6 0	....	....	....	....	....	....	0.18	77	3.2
7.5	0.19	100	3.4	0.19	100	3.4	0.20	98	3.6
8.5	0.26	107	4.7	0.24	107	4.3			
9.5	0.34	112	6.1	0.32	112	5.8			
10.0	....	....	....	....	....	....	0.39	114	7.0
10.5	0.41	118	7.4	0.39	118	7.0			
4″	CHAN	NELS.							
5.0	....	....	....	....	....	....	0.17	60	3.0
5.25	0.18	64	3.2	0.18	64	3.2			
6.0	....	....	....	....	....	....	0.24	67	4.3
6.25	0.25	70	4.5	0.24	70	4.3			
7.25	0.32	77	5.8	0.32	77	5.8			
8.0	....	....	....	....	....	....	0.27	91	4.8
10.0	....	....	....	....	....	....	0.42	104	7.5
4″	ZEES								
8.2	0.25	105							
10.3	0.31	130							
12.4	0.37	155							

Diagram No. 2 4-in. x 7.5-lb. I-Beams



Safe loads given include weight of beam and maximum fiber stress, 16,000 lbs. per sq. in. maximum deflection 1-400th of the span.

THE DIAGRAM ON OPPOSITE PAGE GIVES:

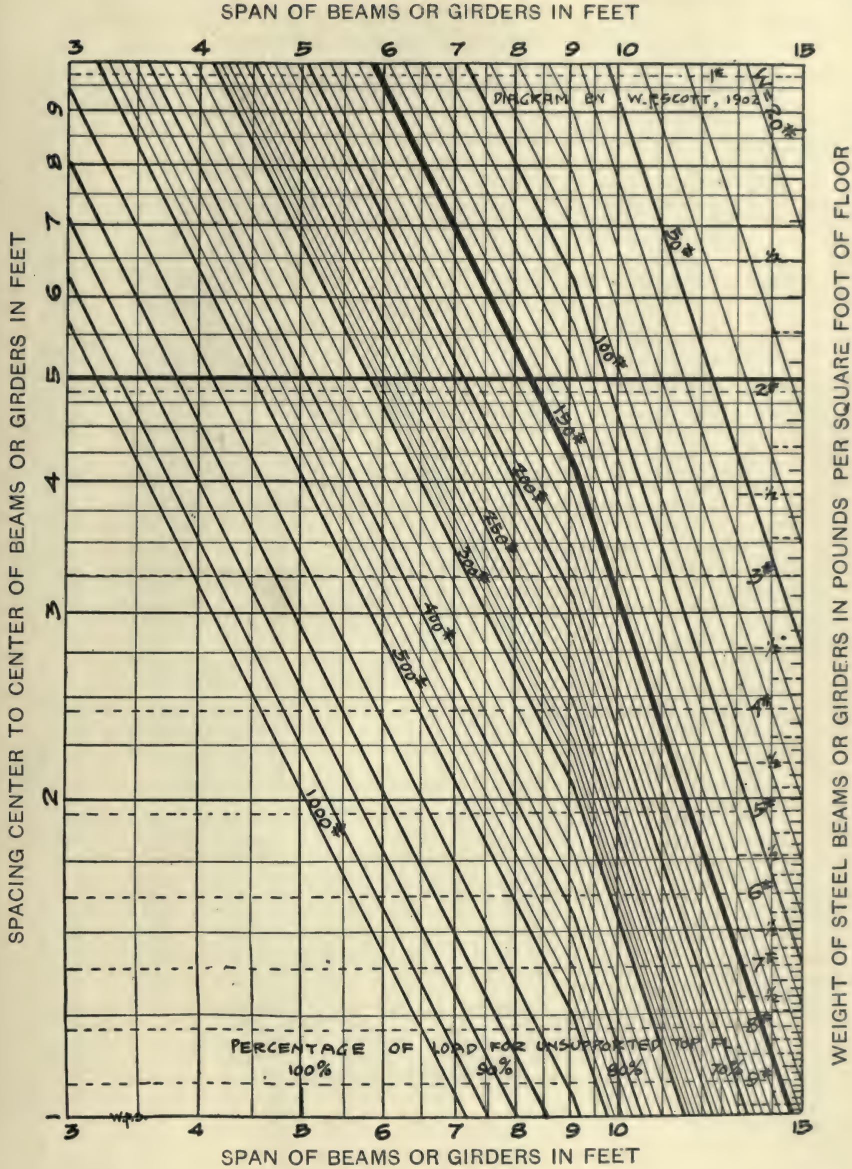
(*a*) The allowable *uniform load* on **5-in. x 9.75 lb. I-beams** in lbs. per sq. ft. of floor.
(*b*) The allowable *spacing* C. to C. in ft. for any span and any uniform load.
(*c*) The allowable *span* in ft. for any uniform load and any spacing.
(*d*) The weight of steel in lbs. per sq. ft. of floor.
(*e*) The percentage of load allowable for any unsupported length of top flange in feet.

THE TABLE FOLLOWING GIVES:

(*f*) The percentage of load allowable on special shapes other than the above standard.
(*g*) The same percentage factor to be used for spacing instead of load.
(*h*) The allowable end reaction for safety of web without reenforcement for buckling.

5″ I Beams	Carnegie, Cambria, Jones & Laughlins, Phœnix			Pencoyd			Passaic		
Weight per foot	Web thickness	Allowable load per square foot	Allowable end reaction	Web thickness	Allowable load per square foot	Allowable end reaction	Web thickness	Allowable load per square foot	Allowable end reaction
Lbs.	Ins.	% W.	Tons	Ins.	% W.	Tons	Ins.	% W.	Tons
9.75	0.21	100	4.7	0.21	100	4.7	0.21	100	4.7
12.	....	....	....	....	....	....	0.34	114	7.6
12.25	0.36	113	8.1	0.34	113	7.6			
13.	....	....	....	....	....	....	0.26	131	5.8
14.75	0.50	127	11.2	0.49	127	11.0			
15.0	....	....	....	....	....	....	0.38	141	8.5
5″	CHAN	NELS							
6.0	....	....	....	....	....	....	0.18	54	4.0
6.5	0.19	63	4.3	0.19	63	4.3			
8.0	....	....	....	....	....	....	0.30	64	6.7
9.0	0.33	73	7.4	0.32	73	7.2	0.25	81	5.6
10.0	....	....	....	....	....	....	0.31	86	7.0
11.5	0.48	88	10.8	0.47	88	10.6			
12.0	....	....	....	....	....	....	0.43	96	9.7
5″	ZEES								
11.6	0.31	111							
13.9	0.37	133							
16.4	0.44	155							
5″	BULB	ANGLE							
10.	0.31	85							

Diagram No. 3 5-in. x 9.75-lb. I-Beams



Safe loads given include weight of beam and maximum fiber stress, 16,000 lbs. per sq. in. maximum deflection 1-400th of the span.

THE DIAGRAM ON OPPOSITE PAGE GIVES:

(*a*) The allowable *uniform load* on **6-in. x 12.25 lb. I-beams** in lbs. per sq. ft. of floor.

(*b*) The allowable *spacing* C. to C. in ft. for any span and any uniform load.

(*c*) The allowable *span* in ft. for any uniform load and any spacing.

(*d*) The weight of steel in lbs. per sq. ft. of floor.

(*e*) The percentage of load allowable for any unsupported length of top flange in feet.

THE TABLE FOLLOWING GIVES:

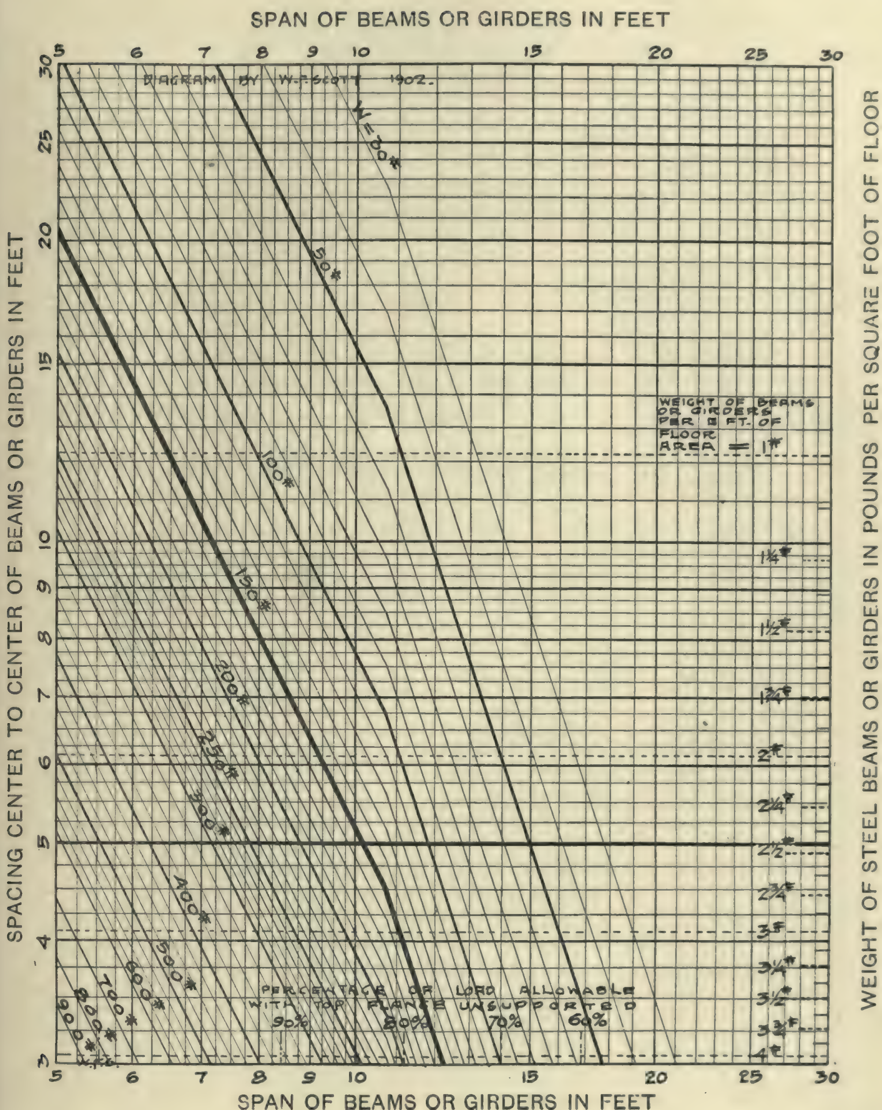
(*f*) The percentage of load allowable on special shapes other than the above standard.

(*g*) The same percentage factor to be used for spacing instead of load.

(*h*) The allowable end reaction for safety of web without reenforcement for buckling.

6″ I Beams	Carnegie, Cambria, Jones & Laughlins, Phœnix			Pencoyd			Passaic		
Weight per foot	Web thickness	Allowable load per square foot	Allowable end reaction	Web thickness	Allowable load per square foot	Allowable end reaction	Web thickness	Allowable load per square foot	Allowable end reaction
Lbs.	Ins.	% W.	Tons	Ins.	% W.	Tons	Ins.	% W.	Tons
12.0	....	....	....	....	....	....	0.22	99	5.9
12.25	0.23	100	6.2	0.23	100	6.2			
14.75	0.35	110	9.4	0.34	111	9.2			
15.0	....	....	....	....	....	....	0.25	121	6.7
17.25	0.48	119	12.9	0.46	122	12.4			
17.5	....	....	....	....	....	....	0.37	131	10.0
20.0	....	....	....	....	....	....	0.50	141	13 5
6″	CHAN	NELS.							
8	0.20	59	5.3	0.20	59	5.3	0.20	58	5.3
9	....	....	....	....	....	....	0.25	62	6.7
10	....	....	....	....	....	....	0.30	66	8.1
10 5	0.32	69	8.6	0.27	75	7.3			
12	....	....	....	....	....	....	0.28	85	7.5
13	0.44	80	11 9	0.40	85	10.8	0.33	89	8.9
15	....	....	....	....	....	....	0.43	97	11.6
15.5	0.56	89	15.1	0.52	96	14.0			
17	....	....	....	....	....	....	0.38	116	10.2
18	....	....	....	....	....	....	0.43	120	11.6
20	....	....	....	....	....	....	0.53	128	14.3
6″	ZEES								
15.6	0.37	115							
18.3	0 44	134							
21.0	0.50	153							
6″	DECK	BEAMS							
14.1	0.28	84							
17.2	0.43	99							
6″	BULB	ANGLES							
12.3	0.31	78							
13.8	0 38	90							
17.2	0.50	104							

Diagram No. 4 **6-in. x 12.25-lb. I-Beams**



Safe loads given include weight of beam and maximum fiber stress, 16,000 lbs. per sq. in. maximum deflection 1-400th of the span.

THE DIAGRAM ON OPPOSITE PAGE GIVES:

(*a*) The allowable *uniform load* on **7-in. x 15 lb. I-beams** in lbs. per sq. ft. of floor.

(*b*) The allowable *spacing* C. to C. in ft. for any span and any uniform load.

(*c*) The allowable *span* in ft. for any uniform load and any spacing.

(*d*) The weight of steel in lbs. per sq. ft. of floor.

(*e*) The percentage of load allowable for any unsupported length of top flange in feet.

THE TABLE FOLLOWING GIVES:

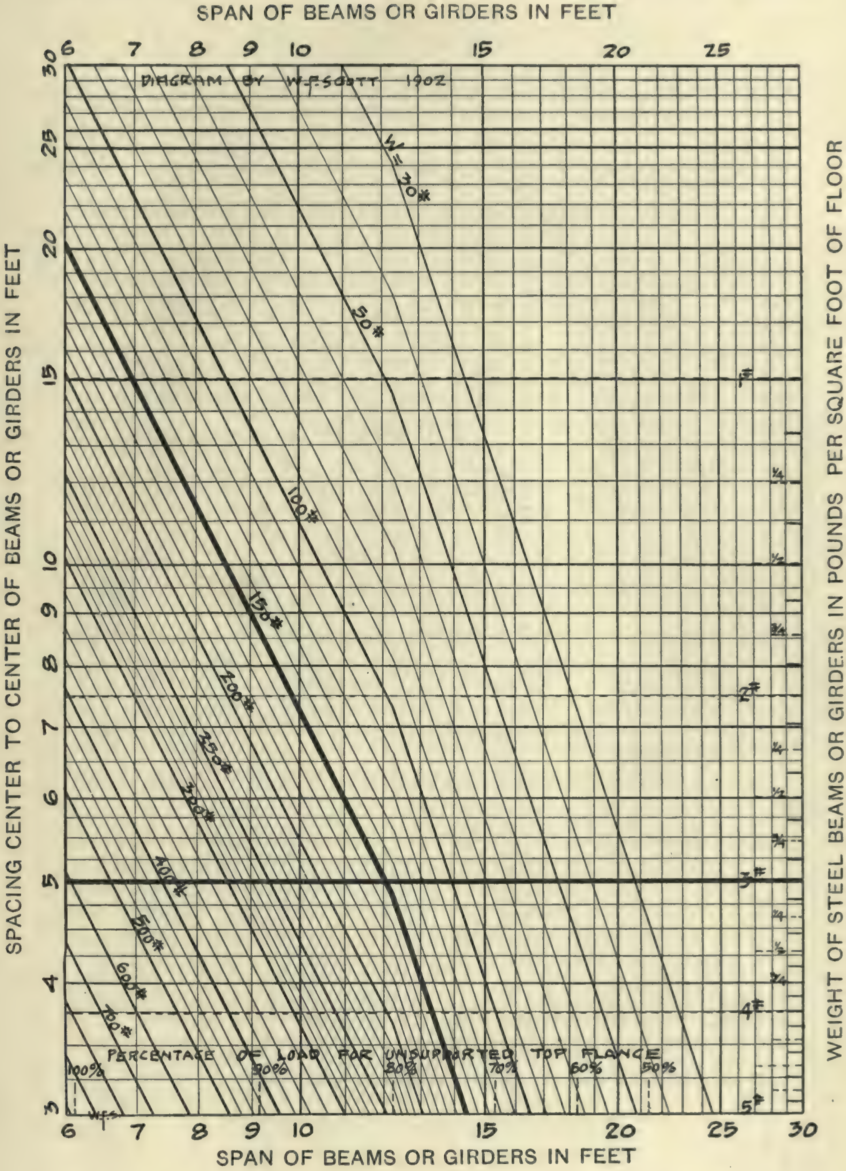
(*f*) The percentage of load allowable on special shapes other than the above standard.

(*g*) The same percentage factor to be used for spacing instead of load.

(*h*) The allowable end reaction for safety of web without reenforcement for buckling.

7″ I Beams	Carnegie, Cambria, Jones & Laughlins, Phœnix			Pencoyd			Passaic		
Weight per foot	Web thickness	Allowable load per square foot	Allowable end reaction	Web thickness	Allowable load per square foot	Allowable end reaction	Web thickness	Allowable load per square foot	Allowable end reaction
Lbs.	Ins.	% W.	Tons	Ins.	% W.	Tons	Ins.	% W.	Tons
15	0.25	100	7.9	0.25	101	7.9	0.23	102	6.9
17.5	0.35	109	11.0	0.34	109	10.7	0.34	111	10.7
20	0.46	116	14.5	0.45	118	14.2	0.28	131	8.8
22	....	....	....	....	....	....	0.36	138	11.3
7″	Chan	nels							
9	....	....	....	....	....	....	0.20	52	5.3
9.75	0.21	58	5.8	0.21	59	5.8			
10	....	....	....	....	....	....	0.24	55	7.4
12	....	....	....	....	....	....	0.33	62	10.4
12.25	0.32	67	10.1	0.31	68	9.8			
13	....	....	....	....	....	....	0.28	75	8.8
14.75	0.42	75	13.2	0.36	82	11.3			
15	....	....	....	....	....	....	0.36	81	11.3
17	....	....	....	....	....	....	0.45	88	14.2
17.25	0.53	83	16.7	0.46	91	14.5			
19.75	0.63	91	19.8	0.57	98	18.0			
7″	Deck	Beams							
18.1	0.31	93							
23.5	0.54	112							
7″	Bulb	Angles							
16	0.34	84							
18.3	0.44	92							

Diagram No. 5 7-in. x 15-lb. I-Beams



Safe loads given include weight of beam and maximum fiber stress, 16,000 lbs. per sq. in. maximum deflection 1-400th of the span.

THE DIAGRAM ON OPPOSITE PAGE GIVES:

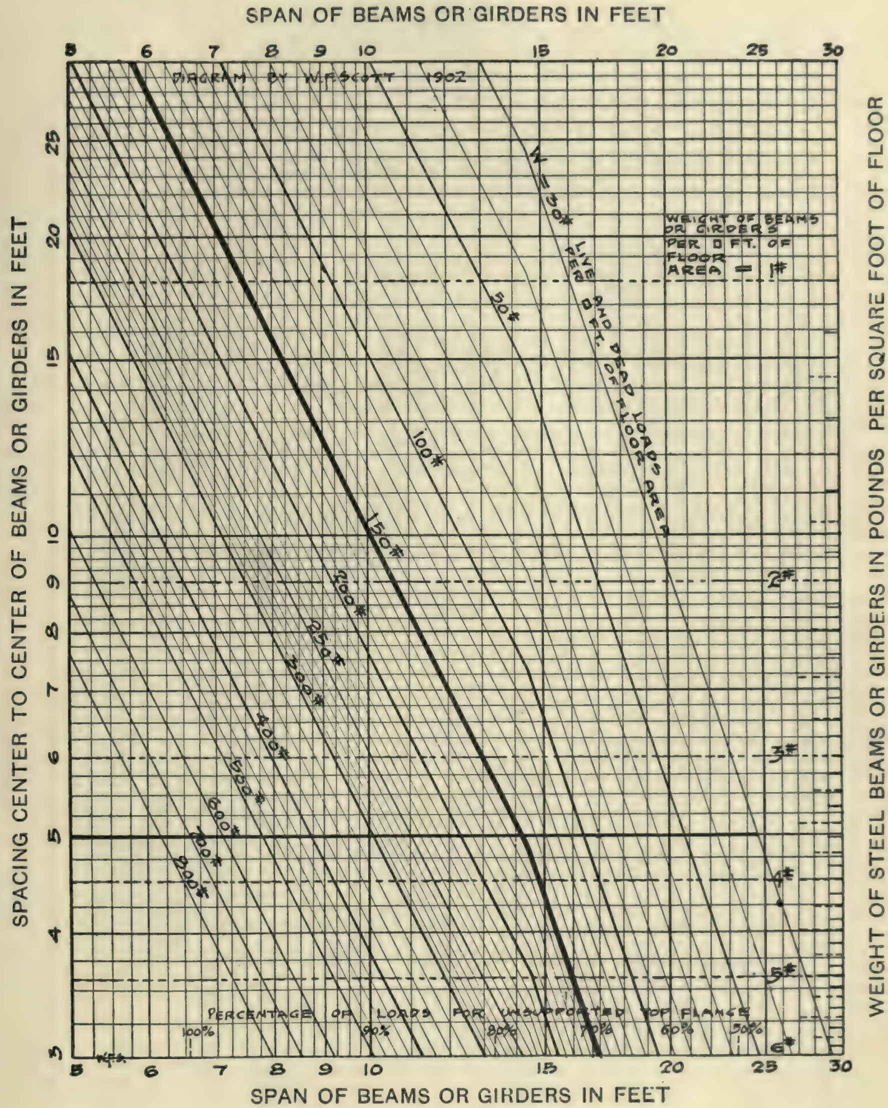
(*a*) The allowable *uniform load* on **8-in. x 18 lb. I-beams** in lbs. per sq. ft. of floor.
(*b*) The allowable *spacing* C. to C. in ft. for any span and any uniform load.
(*c*) The allowable *span* in ft. for any uniform load and any spacing.
(*d*) The weight of steel in lbs. per sq. ft. of floor.
(*e*) The percentage of load allowable for any unsupported length of top flange in feet.

THE TABLE FOLLOWING GIVES:

(*f*) The percentage of load allowable on special shapes other than the above standard.
(*g*) The same percentage factor to be used for spacing instead of load.
(*h*) The allowable end reaction for safety of web without reenforcement for buckling.

8″ I Beam	Carnegie, Cambria, Jones & Laughlins, Phœnix			Pencoyd			Passaic		
Weight per foot	Web thickness	Allowable load per square foot	Allowable end reaction	Web thickness	Allowable load per square foot	Allowable end reaction	Web thickness	Allowable load per square foot	Allowable end reaction
Lbs.	Ins.	% W.	Tons	Ins.	% W.	Tons	Ins.	% W.	Tons
18	0.27	100	9.2	0.27	100	9.2	0.25	100	7.9
20	....	....	....	....	....	....	0.32	105	11.5
20.5	0.36	106	13.0	0.34	107	12.3			
22	....	....	....	....	....	....	0.29	123	10.3
23	0.45	113	16.2	0.44	114	15.9			
25	....	....	....	....	....	....	0.40	131	14.4
25.5	0.54	121	19.5	0.53	122	19.1			
27	....	....	....	....	....	....	0.48	136	17.3
8″	CHAN	NELS							
10	....	....	....	....	....	...	0.20	50	4.5
11	....	....	....	....	....	....	0.24	52	7.2
11.25	0.22	57	5.8	0.22	57	5.8			
12	....	....	....	....	....	....	0.27	56	9.2
13	....	....	....	....	....	....	0.25	63	7.9
13.75	0.31	63	11.2	0.30	64	10.8			
15	....	....	....	....	....	....	0 32	68	11.5
16.25	0.40	71	14.4	0.33	77	11.9			
17	....	....	....	....	....	....	0.40	74	14 4
18.75	0 49	77	17.7	0.42	84	15.1			
21.25	0 58	83	20.9	0.52	91	18.7			
8″	DECK	BEAMS							
20.2	0.31	86							
24.5	0.47	99							
8″	BULB	ANGLE							
19.3	0.41	82							

Diagram No. 6 8-in. x 18-lb. I-Beams



Safe loads given include weight of beam and maximum fiber stress, 16,000 lbs. per sq. in. maximum deflection 1-400th of the span.

THE DIAGRAM ON OPPOSITE PAGE GIVES:

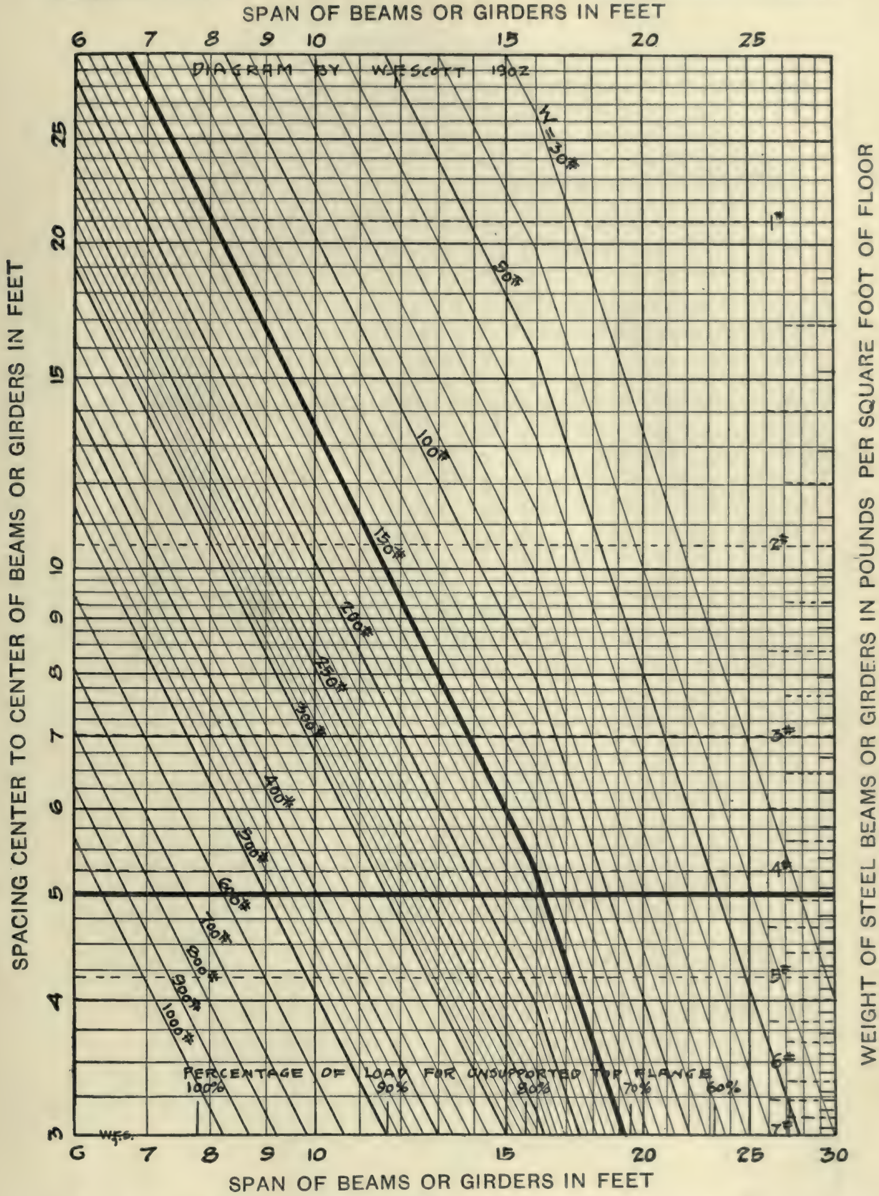
(*a*) The allowable *uniform load* on **9-in. x 21 lb. I-beams** in lbs. per sq. ft. of floor.
(*b*) The allowable *spacing* C. to C. in ft. for any span and any uniform load.
(*c*) The allowable *span* in ft. for any uniform load and any spacing.
(*d*) The weight of steel in lbs. per sq. ft. of floor.
(*e*) The percentage of load allowable for any unsupported length of top flange in feet.

THE TABLE FOLLOWING GIVES:

(*f*) The percentage of load allowable on special shapes other than the above standard.
(*g*) The same percentage factor to be used for spacing instead of load.
(*h*) The allowable end reaction for safety of web without reenforcement for buckling.

9″ I Beam	Carnegie, Cambria, Jones & Laughlins, Phœnix			Pencoyd			Passaic		
Weight per foot	Web thickness	Allowable load per square foot	Allowable end reaction	Web thickness	Allowable load per square foot	Allowable end reaction	Web thickness	Allowable load per square foot	Allowable end reaction
Lbs.	Ins.	% W.	Tons	Ins.	% W.	Tons	Ins.	% W.	Tons
21	0.29	100	10.5	0.29	100	10.5	0.27	99	9.1
23.3	....	....	....	....	....	....	0.35	105	14.2
25	0.41	108	16.6	0.39	109	15.8	0.40	108	16.2
27	....	....	....	....	....	....	0.31	130	11.9
30	0.57	120	23.1	0.56	120	22.7	0.41	137	16.6
33	....	....	....	....	....	....	0.51	144	20.7
35	0.73	132	29.6	0.72	133	29.2			
9″	CHANNELS								
13	....	....	....	....	....	....	0.23	53	6.0
13.25	0.23	56	6.0	0.23	56	6.0			
14	....	....	....	....	....	....	0.26	55	8.2
15	0.29	59	10.5	0.28	60	9.8	0.30	58	11.1
16	....	....	....	....	....	....	0.28	67	9.8
18	....	....	....	....	....	....	0.35	72	14.2
20	0.45	71	18.2	0.38	79	15.4			
21	....	....	....	....	....	....	0.45	79	18.2
25	0.62	83	25.1	0.54	90	21.9			
9″	DECK BEAM								
26	0.44	94							
30	0.57	104							
9″	BULB ANGLE								
21.8	0.44	77							

Diagram No. 7 9-in. x 21-lb. I-Beams



Safe loads given include weight of beam and maximum fiber stress, 16,000 lbs. per sq. in. maximum deflection 1-400th of the span.

THE DIAGRAM ON OPPOSITE PAGE GIVES:

(*a*) The allowable *uniform load* on **10-in. x 25 lb. I-beams** in lbs. per sq. ft. of floor.

(*b*) The allowable *spacing* C. to C. in ft. for any span and any uniform load.

(*c*) The allowable *span* in ft. for any uniform load and any spacing.

(*d*) The weight of steel in lbs. per sq. ft. of floor.

(*e*) The percentage of load allowable for any unsupported length of top flange in feet.

THE TABLE FOLLOWING GIVES:

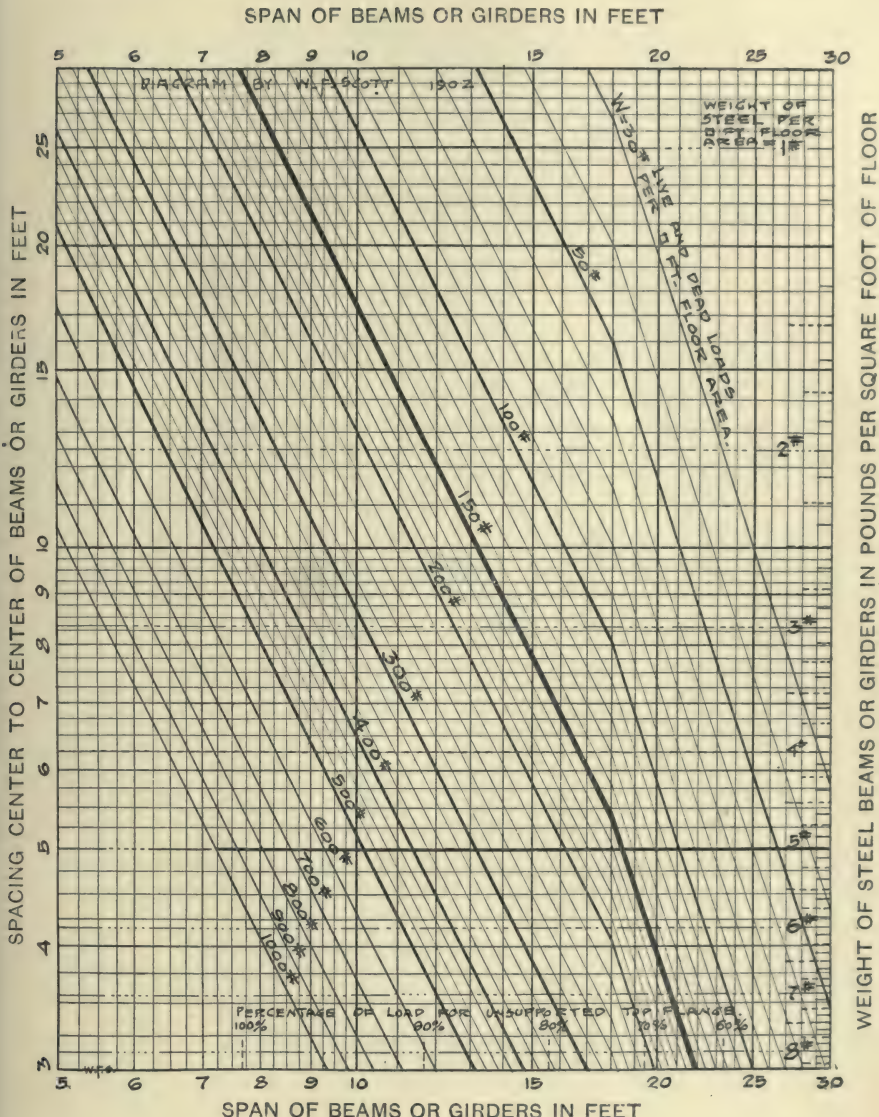
(*f*) The percentage of load allowable on special shapes other than the above standard.

(*g*) The same percentage factor to be used for spacing instead of load.

(*h*) The allowable end reaction for safety of web without reenforcement for buckling.

10″ I Beams	Carnegie, Cambria, Jones & Laughlins, Phœnix			Pencoyd			Passaic		
Weight per foot	Web thickness	Allowable load per square foot	Allowable end reaction	Web thickness	Allowable load per square foot	Allowable end reaction	Web thickness	Allowable load per square foot	Allowable end reaction
Lbs.	Ins.	% W.	Tons	Ins.	% W.	Tons	Ins.	% W.	Tons
25	0.31	100	12.0	0.31	100	12.0	0.31	100	12.0
27	....	....	....	....	....	....	0.37	104	16.4
30	0.46	110	20.7	0.44	110	19.8	0.45	110	20.2
33	....	....	....	....	....	....	0 37	132	16.4
35	0.60	120	27.0	0.44	134	19.8	0.43	136	19.4
40	0.75	130	33.8	0.59	143	26.6	0.58	146	26.1
10″	CHANN	ELS							
15	0.24	55	6.0	0.24	55	6.0	0.25	54	6.8
17	....	....	....	....	....	....	0.29	58	10.2
18	....	....	....	....	....	....	0.32	60	12.7
20	0.38	64	17.1	0.38	64	17.1	0.31	70	12.0
25	0.53	74	23.9	0.45	81	20.2	0 46	83	20.7
30	0.68	84	30.6	0.60	92	27.0	0.60	90	27.0
35	0.82	94	36.9	0.75	102	33.8			
10″	DECK	BEAMS							
27.3	0.38	88							
35.7	0.63	105							
10″	BULB	ANGLES							
26.5	0.48	82							
32.0	0.63	89							

Diagram No. 8 **10-in. x 25-lb. I-Beams**



Safe loads given include weight of beam and maximum fiber stress, 16,000 lbs. per sq. in. maximum deflection 1-400th of the span.

THE DIAGRAM ON OPPOSITE PAGE GIVES:

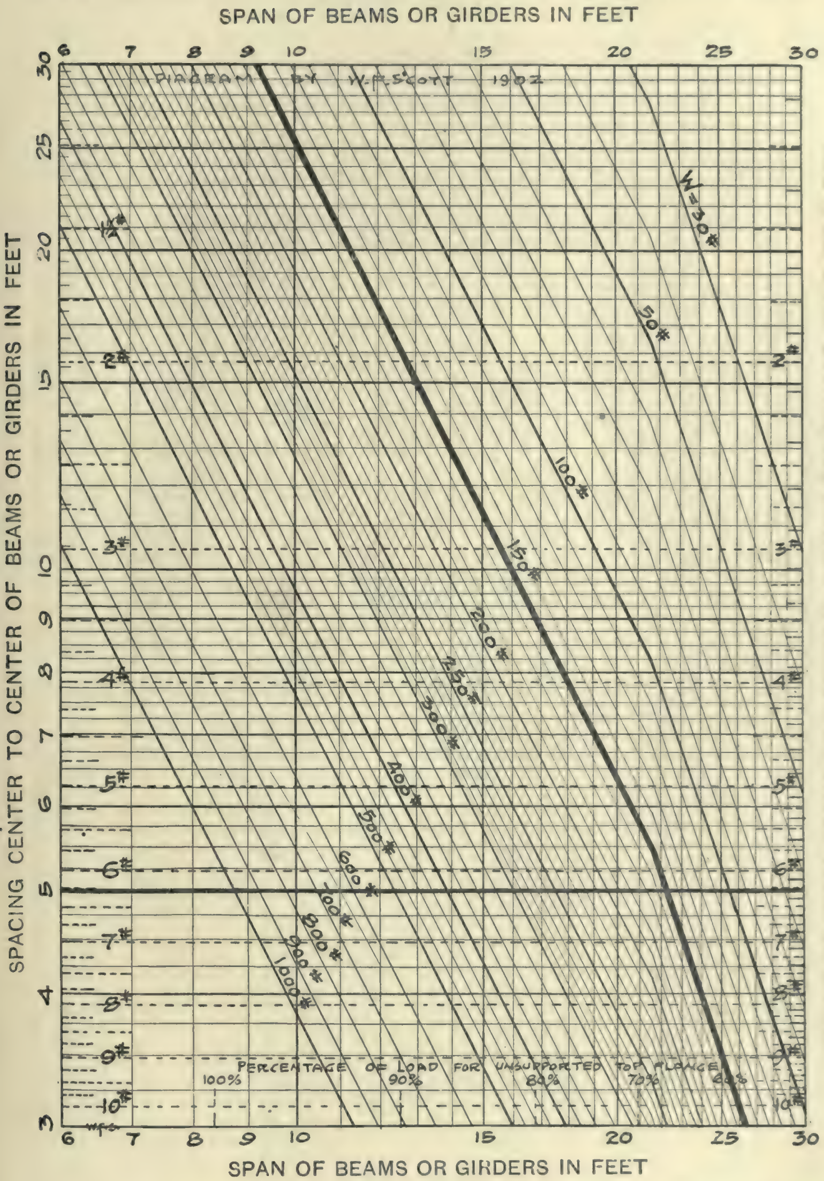
(*a*) The allowable *uniform load* on **12-in. x 31.5 lb. I-beams** in lbs. per sq. ft. of floor.
(*b*) The allowable *spacing* C. to C. in ft. for any span and any uniform load.
(*c*) The allowable *span* in ft. for any uniform load and any spacing.
(*d*) The weight of steel in lbs. per sq. ft. of floor.
(*e*) The percentage of load allowable for any unsupported length of top flange in feet.

THE TABLE FOLLOWING GIVES:

(*f*) The percentage of load allowable on special shapes other than the above standard.
(*g*) The same percentage factor to be used for spacing instead of load.
(*h*) The allowable end reaction for safety of web without reenforcement for buckling.

12″ I Beams	Carnegie, Cambria, Jones & Laughlins, Phœnix			Pencoyd			Passaic		
Weight per foot	Web thickness	Allowable load per square foot	Allowable end reaction	Web thickness	Allowable load per square foot	Allowable end reaction	Web thickness	Allowable load per square foot	Allowable end reaction
Lbs.	Ins.	% W.	Tons	Ins.	% W.	Tons	Ins.	% W.	Tons
31.5	0.35	100	15.6	0.35	101	15.6	0.35	101	15.6
35	0.44	106	23.8	0.42	107	22.1	0.44	107	28.8
12″	CHAN	NELS							
20	....	....	....	....	....	....	0.28	58	8.5
20.5	0.28	60	8.5	0.28	60	8.5			
23	....	....	....	....	....	....	0.35	63	15.6
25	0.39	67	19.3	0.39	67	19.3	0.40	66	20.3
27	....	....	....	....	....	....	0.38	74	18.4
30	0.51	75	27.6	0.51	75	27.6	0.45	79	24.3
33	....	....	....	....	....	....	0.53	84	28.6
35	0.64	83	34.6	0.50	96	27.0	0.58	88	31.3
40	0.76	91	41.1	0.62	104	33.5			
11½″	DECK	BEAMS							
32.2	0.42	76							
37.0	0.55	85							

Diagram No. 9 **12-in. x 31.5-lb. I-Beams**



Safe loads given include weight of beam and maximum fiber stress, 16,000 lbs. per sq. in. maximum deflection 1-400th of the span.

THE DIAGRAM ON OPPOSITE PAGE GIVES:

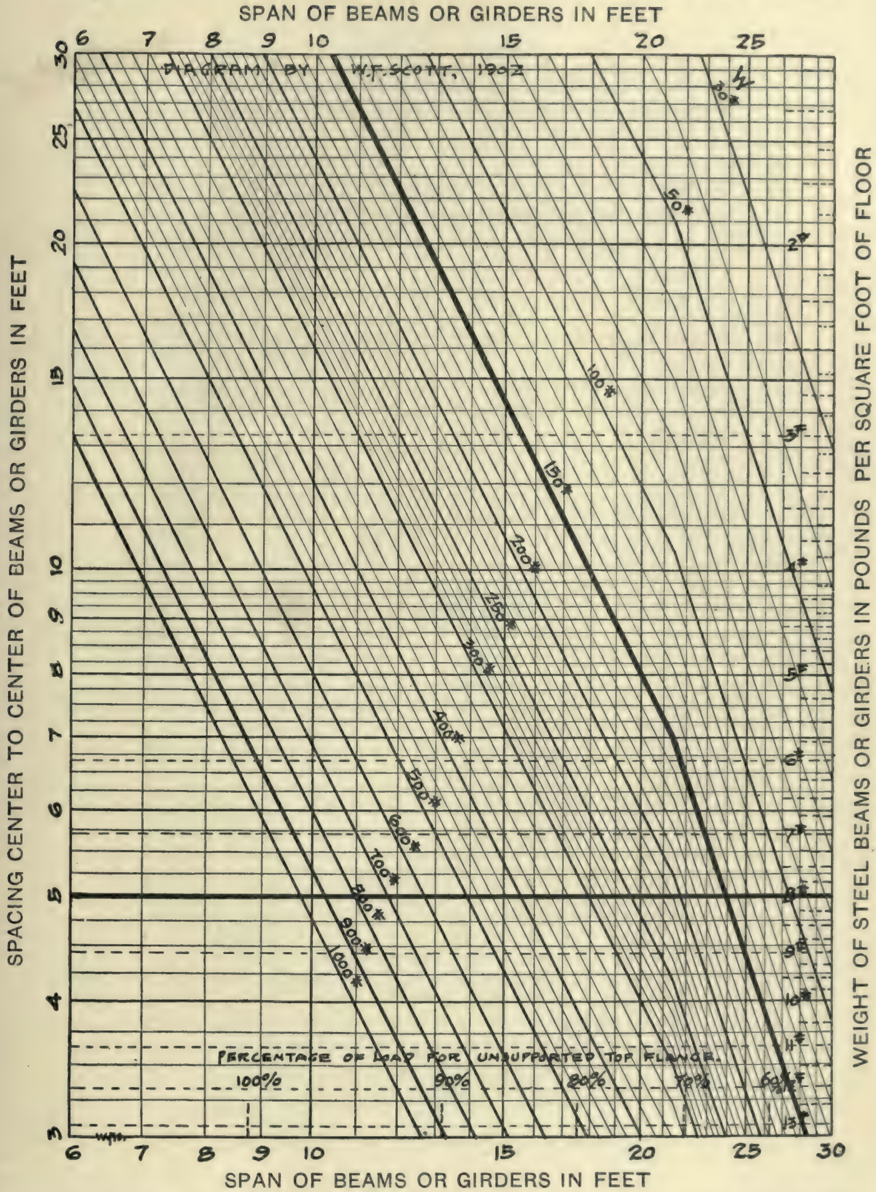
(*a*) The allowable *uniform load* on **12-in. x 40 lb. I-beams** in lbs. per sq. ft. of floor.
(*b*) The allowable *spacing* C. to C. in ft. for any span and any uniform load.
(*c*) The allowable *span* in ft. for any uniform load and any spacing.
(*d*) The weight of steel in lbs. per sq. ft. of floor.
(*e*) The percentage of load allowable for any unsupported length of top flange in feet.

THE TABLE FOLLOWING GIVES:

(*f*) The percentage of load allowable on special shapes other than the above standard.
(*g*) The same percentage factor to be used for spacing instead of load.
(*h*) The allowable end reaction for safety of web without reenforcement for buckling.

12" I Beams	Carnegie, Cambria, Jones & Laughlins, Phœnix			Pencoyd			Passaic		
Weight per foot	Web thickness	Allowable load per square foot	Allowable end reaction	Web thickness	Allowable load per square foot	Allowable end reaction	Web thickness	Allowable load per square foot	Allowable end reaction
Lbs.	Ins.	% W.	Tons	Ins.	% W.	Tons	Ins.	% W.	Tons
40	0.46	100	24.9	0.42	102	22.1	0.39	104	19.3
45	0.58	106	31.3	0.54	109	29.2	0.51	110	27.6
50	0.70	113	37.8	0.55	123	29.7	0.64	118	34.6
55	0.82	119	44.3	0.56	137	30.3	0.63	133	34.0
60	....	....	....	0.68	143	36.8	0.75	139	40.5
65	....	....	....	0.80	150	43.2	0.88	146	47.5
12"	CHAN	NELS							
20	....	....	....	....	....	....	0.28	46	8.5
20.5	0.28	48	8.5	0.28	48	8.5			
23	....	....	....	....	....	....	0.35	50	15.6
25	0.39	54	19.3	0.39	54	19.3	0.40	53	20.3
27	....	....	....	....	....	....	0.38	60	18.4
30	0.51	60	27.6	0.51	60	27.6	0.45	64	24.3
33	....	....	....	.. .	.. .	....	0.53	68	28.6
35	0.64	67	34.6	0.50	77	27.0	0.58	70	31.3
40	0.76	73	41.1	0.62	83	33.5			

Diagram No. 10 **12-in. x 40-lb. I-Beams**



Safe loads given include weight of beam and maximum fiber stress, 16,000 lbs. per sq. in. maximum deflection 1-400th of the span.

THE DIAGRAM ON OPPOSITE PAGE GIVES:

(*a*) The allowable *uniform load* on **15-in. x 42 lb. I-beams** in lbs. per sq. ft. of floor.

(*b*) The allowable *spacing* C. to C. in ft. for any span and any uniform load.

(*c*) The allowable *span* in ft. for any uniform load and any spacing.

(*d*) The weight of steel in lbs. per sq. ft. of floor.

(*e*) The percentage of load allowable for any unsupported length of top flange in feet.

THE TABLE FOLLOWING GIVES:

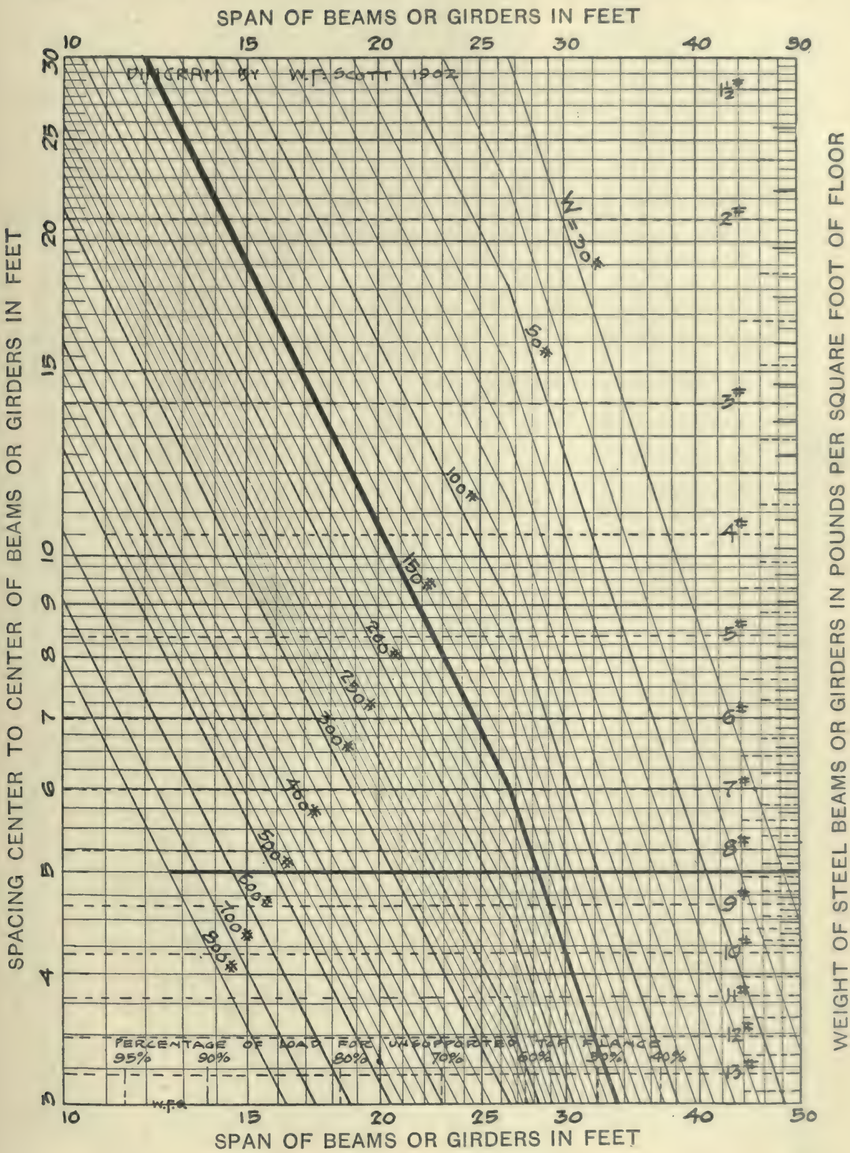
(*f*) The percentage of load allowable on special shapes other than the above standard.

(*g*) The same percentage factor to be used for spacing instead of load.

(*h*) The allowable end reaction for safety of web without reenforcement for buckling.

15″ I Beams	Carnegie, Cambria, Jones & Laughlins, Phœnix			Pencoyd			Passaic		
Weight per foot	Web thickness	Allowable load per square foot	Allowable end reaction	Web thickness	Allowable load per square foot	Allowable end reaction	Web thickness	Allowable load per square foot	Allowable end reaction
Lbs.	Ins.	% W.	Tons	Ins.	% W.	Tons	Ins.	% W.	Tons
42	0.41	100	19.1	0.41	100	19.1	0.40	97	17.7
45	0.46	103	25.5	0.45	104	24.3	0.46	101	25.5
50	0.56	109	37.2	0.48	117	28.1	0.45	120	24.3
55	0.66	116	44.6	0.58	123	39.2	0.55	126	36.2
15″	CHANNELS								
33	0.40	70	17.7	0.40	71	17.7	0.40	69	17.7
35	0.43	72	21.8	0.42	73	20.3	0.44	71	23.0
40	0.52	79	32.8	0.52	79	32.8	0.54	77	34.8
45	0.62	85	41.9	0.62	85	41.9	0.64	84	43.2
50	0.72	91	48.7	0.63	100	42.6	0.73	90	49.3
55	0.82	98	55.4	0.72	106	48.7			

Diagram No. 11 **15-in. x 42-lb. I-Beams**



Safe loads given include weight of beam and maximum fiber stress, 16,000 lbs. per sq. in. maximum deflection 1-400th of the span.

THE DIAGRAM ON OPPOSITE PAGE GIVES:

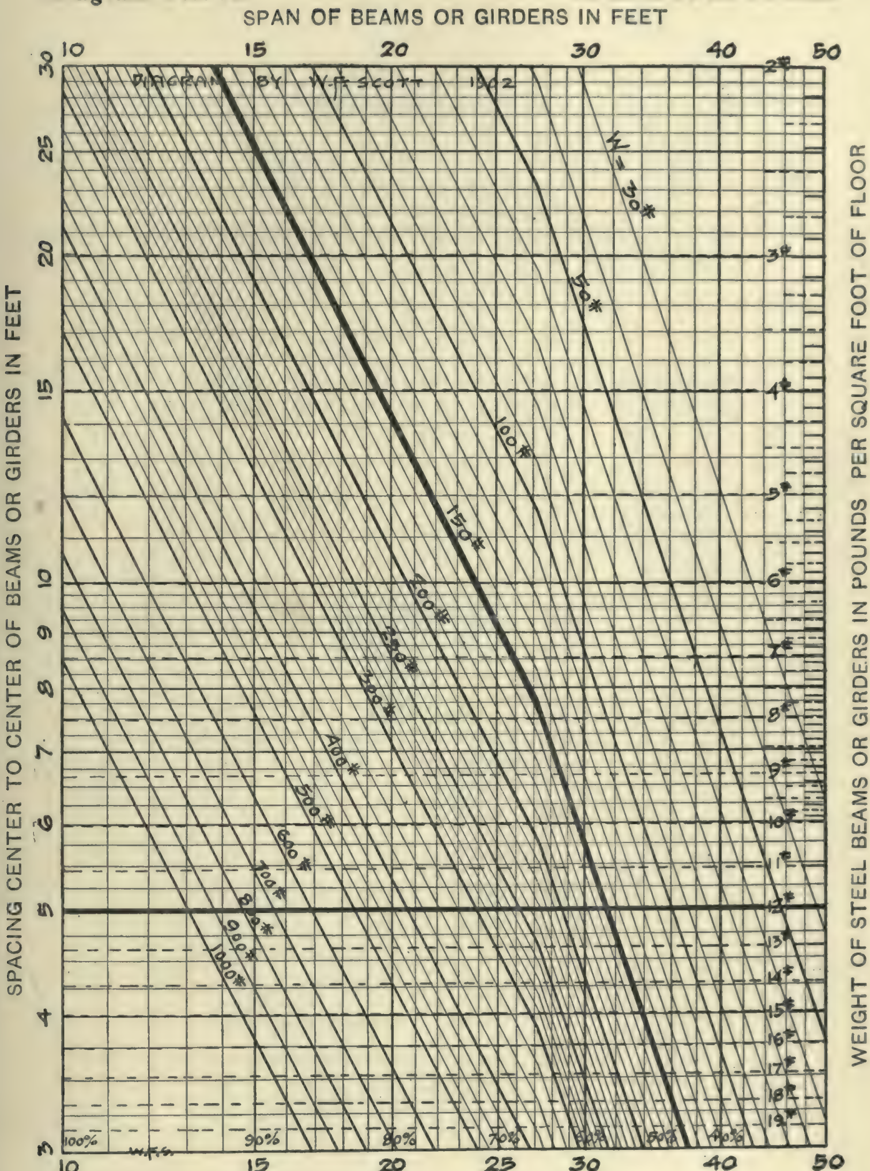
(*a*) The allowable *uniform load* on **15-in. x 60 lb. I-beams** in lbs. per sq. ft. of floor.
(*b*) The allowable *spacing* C. to C. in ft. for any span and any uniform load.
(*c*) The allowable *span* in ft. for any uniform load and any spacing.
(*d*) The weight of steel in lbs. per sq. ft. of floor.
(*e*) The percentage of load allowable for any unsupported length of top flange in feet.

THE TABLE FOLLOWING GIVES:

(*f*) The percentage of load allowable on special shapes other than the above standard.
(*g*) The same percentage factor to be used for spacing instead of load.
(*h*) The allowable end reaction for safety of web without reenforcement for buckling.

15″ I Beams	Carnegie, Cambria, Jones & Laughlins, Phœnix			Pencoyd			Passaic		
Weight per foot	Web thickness	Allowable load per square foot	Allowable end reaction	Web thickness	Allowable load per square foot	Allowable end reaction	Web thickness	Allowable load per square foot	Allowable end reaction
Lbs.	Ins.	% W.	Tons	Ins.	% W.	Tons	Ins.	% W.	Tons
60	0.59	100	39.9	0.55	102	36.2	0.52	105	32.8
65	0.69	104	46.6	0.65	106	43.9	0.62	109	41.9
70	0.78	109	52.7	0.63	118	42.6	0.72	114	48.7
75	0.88	114	59.4	0.73	122	49.3	0.81	118	54.7
80	0.81	131	54.7	0.83	127	56.1	0.91	123	61.5
85	0.89	134	60.1						
90	0.99	139	66.9						
95	1.09	143	73.6						
100	1.18	148	79.7						
15″	CHANNELS								
33	0.40	51	17.7	0.40	51	17.7	0.40	50	17.7
35	0.43	53	21.8	0.42	53	20.3	0.44	52	23.0
40	0.52	57	32.8	0.52	57	32.8	0.54	56	34.8
45	0.62	62	41.9	0.62	62	41.9	0.64	61	43.2
50	0.72	66	48.7	0.63	72	42.6	0.73	65	49.3
55	0.82	71	55.4	0.72	77	48.7			

Diagram No. 12 15-in. x 60-lb. I-Beams



Safe loads given include weight of beam and maximum fiber stress, 16,000 lbs. per sq. in. maximum deflection 1-400th of the span.

THE DIAGRAM ON OPPOSITE PAGE GIVES:

(*a*) The allowable *uniform load* on **18-in. x 55 lb. I-beams** in lbs. per sq. ft. of floor.

(*b*) The allowable *spacing* C. to C. in ft. for any span and any uniform load.

(*c*) The allowable *span* in ft. for any uniform load and any spacing.

(*d*) The weight of steel in lbs. per sq. ft. of floor.

(*e*) The percentage of load allowable for any unsupported length of top flange in feet.

THE TABLE FOLLOWING GIVES:

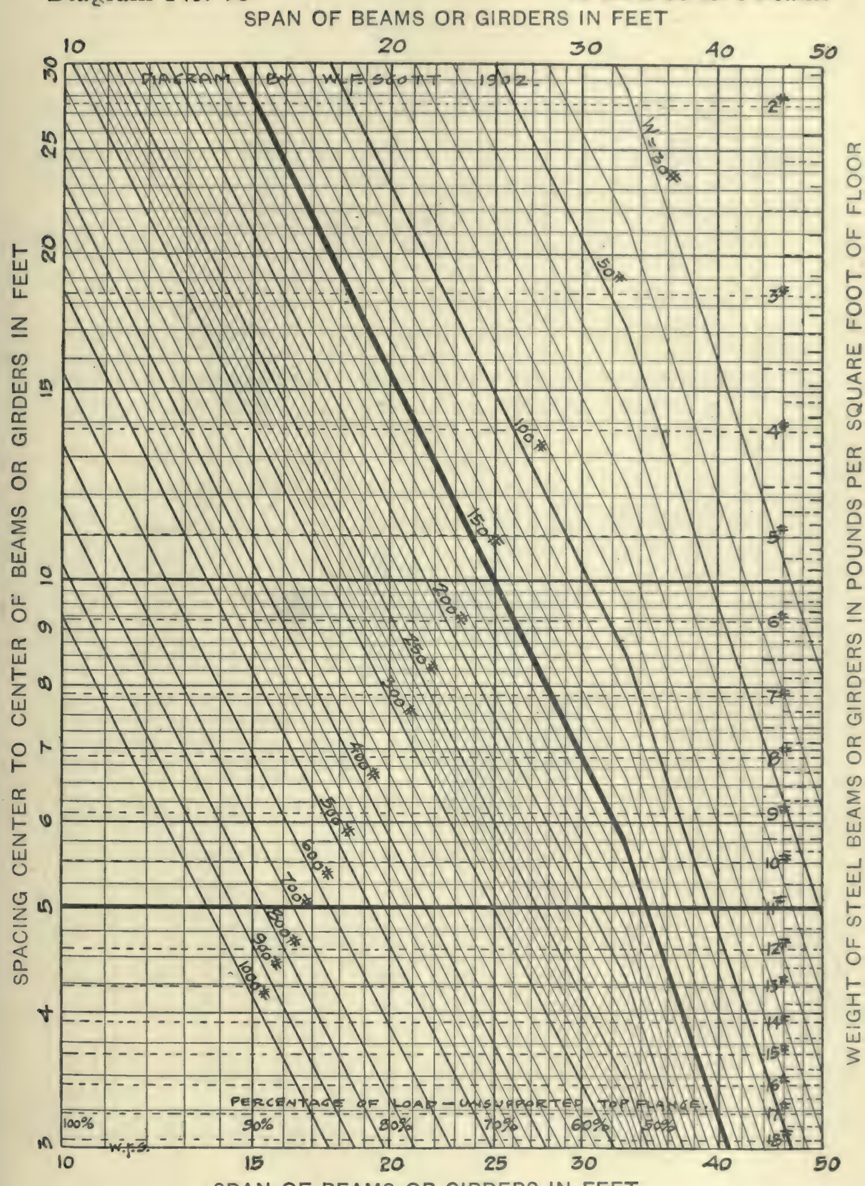
(*f*) The percentage of load allowable on special shapes other than the above standard.

(*g*) The same percentage factor to be used for spacing instead of load.

(*h*) The allowable end reaction for safety of web without reenforcement for buckling.

18″ I Beams	Carnegie, Cambria, Jones & Laughlins, Phœnix			Pencoyd			Passaic		
Weight per foot	Web thickness	Allowable load per square foot	Allowable end reaction	Web thickness	Allowable load per square foot	Allowable end reaction	Web thickness	Allowable load per square foot	Allowable end reaction
Lbs.	Ins.	% W.	Tons	Ins.	% W.	Tons	Ins.	% W.	Tons
55	0.46	100	22.4	0.46	101	22.4	0.47	101	23.8
60	0.55	106	36.2	0.54	107	34.5	0.55	106	36.2
65	0.64	111	49.4	0.63	112	47.7	0.64	111	49.4
70	0.72	116	58.3	0.62	123	46.2	0.65	122	50.5
75	....	...	....	0.71	128	57.5	0.62	137	46.2
80	....	....	....	0.79	133	64.0	0.70	142	56.7
85	....	....	....	0.74	144	59.9			
90	....	....	....	0.82	149	66.4			

Diagram No. 13 **18-in. x 55-lb. I-Beams**



Safe loads given include weight of beam and maximum fiber stress, 16,000 lbs. per sq. in. maximum deflection 1-400th of the span.

THE DIAGRAM ON OPPOSITE PAGE GIVES:

(*a*) The allowable *uniform load* on **20-in. x 65 lb. I-beams** in lbs. per sq. ft. of floor.

(*b*) The allowable *spacing* C. to C. in ft. for any span and any uniform load.

(*c*) The allowable *span* in ft. for any uniform load and any spacing.

(*d*) The weight of steel in lbs. per sq. ft. of floor.

(*e*) The percentage of load allowable for any unsupported length of top flange in feet.

THE TABLE FOLLOWING GIVES:

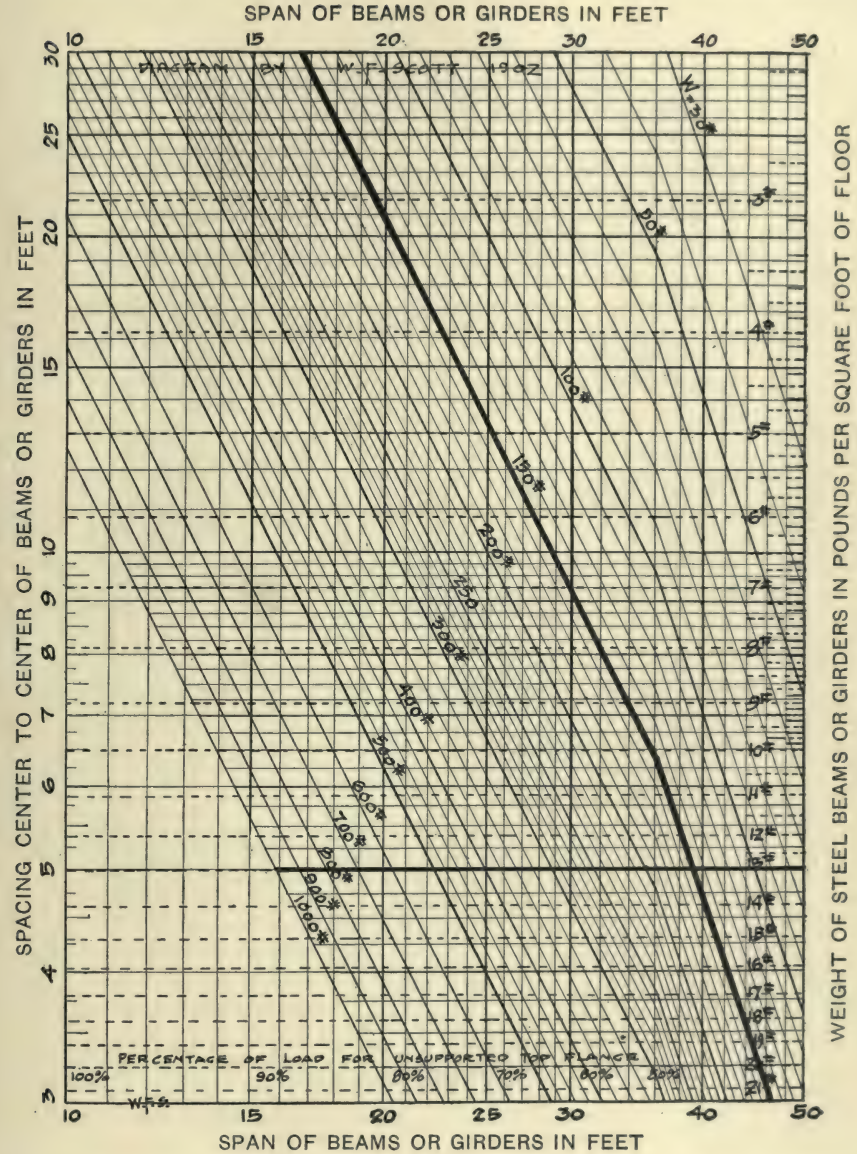
(*f*) The percentage of load allowable on special shapes other than the above standard.

(*g*) The same percentage factor to be used for spacing instead of load.

(*h*) The allowable end reaction for safety of web without reenforcement for buckling.

20″ I Beams	Carnegie, Cambria, Jones & Laughlins, Phœnix			Pencoyd			Passaic		
Weight per foot	Web thickness	Allowable load per square foot	Allowable end reaction	Web thickness	Allowable load per square foot	Allowable end reaction	Web thickness	Allowable load per square foot	Allowable end reaction
Lbs.	Ins.	% W.	Tons	Ins.	% W.	Tons	Ins.	% W.	Tons
65	0.50	100	25.0	0.50	100	25.0	0.50	99	25.0
70	0.57	104	38.5	0.56	105	38.5	0.57	102	38.5
75	0.65	109	50.5	0.64	109	48.6	0.66	106	51.7
80	0.60	125	41.8	0.63	120	46.6	0.69	115	56.6
85	0.66	129	51.7	0.70	124	54.5	0.76	119	66.9
90	0.74	133	64.0	0.78	128	57.8	0.78	129	70.2
95	0.81	137	72.9	0.74	137	68.5			
100	0.88	142	79.2	0.81	141	76.5			

Diagram No. 14 **20-in. x 65-lb. I-Beams**



Safe loads given include weight of beam and maximum fiber stress, 16,000 lbs. per sq. in. maximum deflection 1-400th of the span.

THE DIAGRAM ON OPPOSITE PAGE GIVES:

(*a*) The allowable *uniform load* on **24-in. x 80 lb. I-beams** in lbs. per sq. ft. of floor.

(*b*) The allowable *spacing* C. to C. in ft. for any span and any uniform load.

(*c*) The allowable *span* in ft. for any uniform load and any spacing.

(*d*) The weight of steel in lbs. per sq. ft. of floor.

(*e*) The percentage of load allowable for any unsupported length of top flange in feet.

THE TABLE FOLLOWING GIVES:

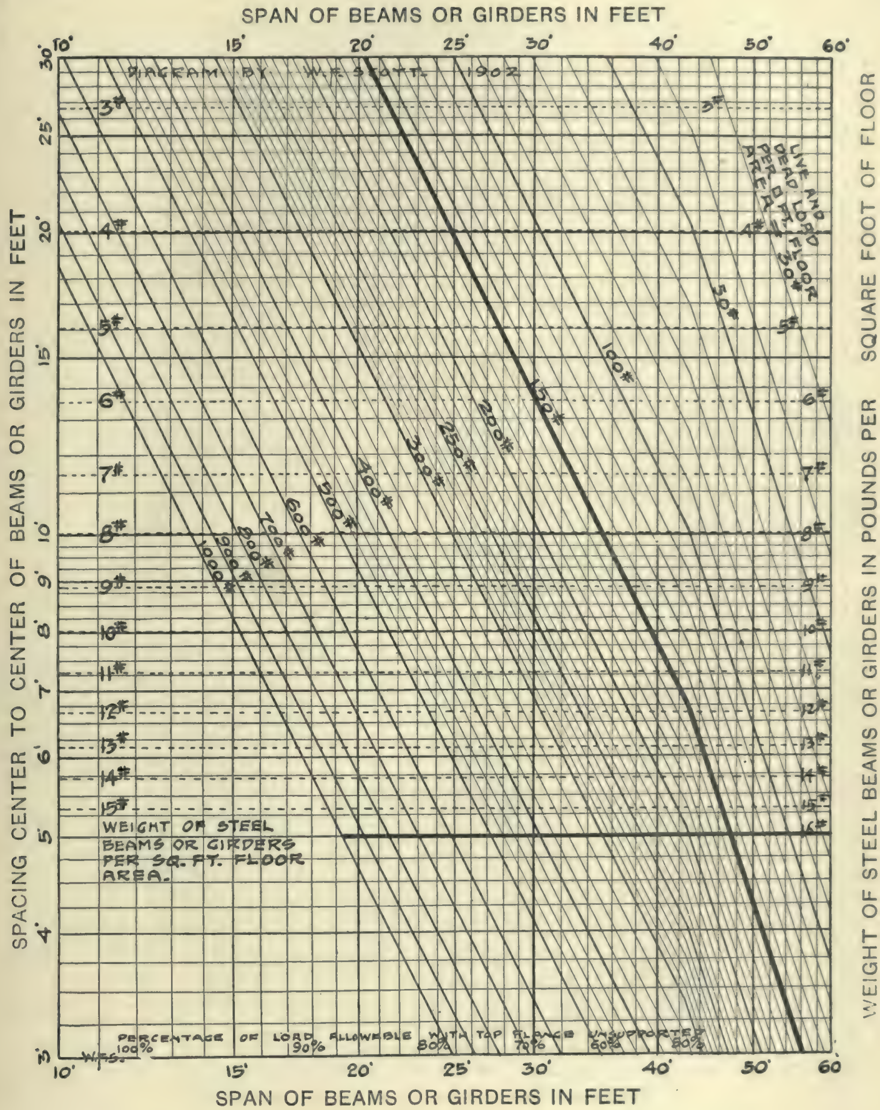
(*f*) The percentage of load allowable on special shapes other than the above standard.

(*g*) The same percentage factor to be used for spacing instead of load.

(*h*) The allowable end reaction for safety of web without reenforcement for buckling.

24″ I Beams	Carnegie, Cambria, Jones & Laughlins, Phœnix			Pencoyd			Passaic		
Weight per foot	Web thickness	Allowable load per square foot	Allowable end reaction	Web thickness	Allowable load per square foot	Allowable end reaction	Web thickness	Allowable load per square foot	Allowable end reaction
Lbs.	Ins.	% W.	Tons	Ins.	% W.	Tons	Ins.	% W.	Tons
80	0.50	100	20.4	0.50	101	20.4			
85	0.57	104	30.9	0.56	105	24.9			
90	0.63	107	42.3	0.56	113	29.4			
95	0.69	111	54.5	0.62	116	34.0			
100	0.75	114	66.3	0.68	120	39.9			

Diagram No. 15 **24-in. x 80-lb. I-Beams**



Safe loads given include weight of beam and maximum fiber stress, 16,000 lbs. per sq. in. maximum deflection 1-400th of the span.

Diagrams Nos. 16, 17, 18, 19, 20 and 21

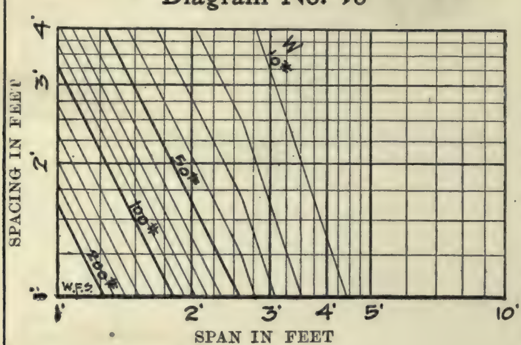
FOR GIVING:

(*a*) The allowable uniformly distributed live and dead load, on **Angles** and **Tees,** as beams or girders, in pounds per square foot of floor.

(*b*) The allowable spacing, center to center, for any span and any uniform loading.

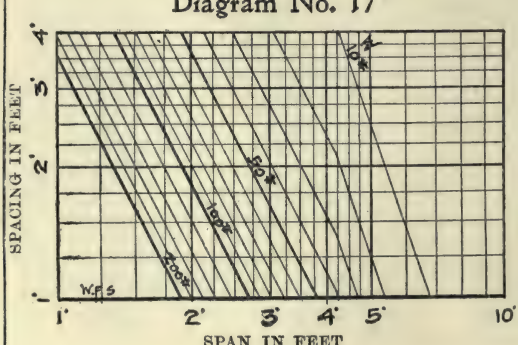
(*c*) The allowable span in feet for any uniform loading and any spacing.

Diagram No. 16



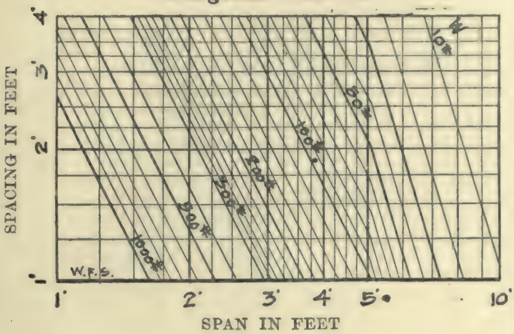
1	2	3	4	5	6
Section.	Width of leg or flange	Height of leg or stem	Thickness	Weight per lineal foot	Allowable load per sq. ft
	Ins	Ins	Ins	Lbs.	% W.
Angles	¾	¾	⅛	0.6	55
			$\frac{3}{16}$	0.8	77
	1	1	⅛	0.8	100
			$\frac{3}{16}$	1.2	142
			¼	1.5	180
Tees	1	1		0.87	97
				1.23	160

Diagram No. 17



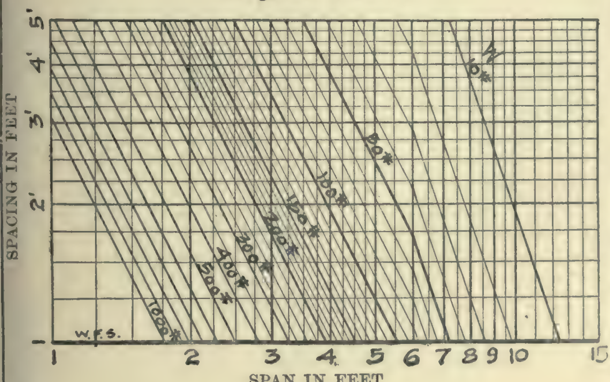
1	2	3	4	5	6
	Ins	Ins	Ins	Lbs.	% W.
Angles	1¼	1¼	⅛	1.0	70
			$\frac{3}{16}$	1.5	100
			¼	1.9	130
			$\frac{5}{16}$	2.4	156
	1½	1½	⅛	1.2	100
			$\frac{3}{16}$	1.8	150
			¼	2.4	190
			$\frac{5}{16}$	2.9	230
			⅜	3.4	270
Tees	1¼	1¼		1.53	100
	1¾	1¼		3.6	214
	2¼	1¼		2.9	129
	1½	1½		1.84	157
	2	1½		2.4	200

Diagram No. 18



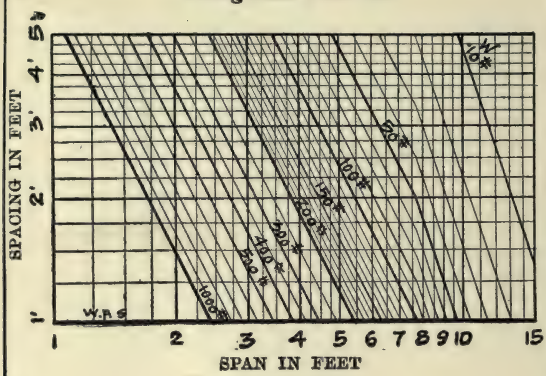
1	2	3	4	5	6
	Ins	Ins	Ins	Lbs.	% W.
Angles	1¾	1¾	3/16	2.1	56
			¼	2.8	76
			5/16	3.4	92
			⅜	4.0	104
	2	2	3/16	2.5	76
			¼	3.2	100
			5/16	4.0	120
			⅜	4.7	140
Tees	1¾	1¾		3.1	76
	2	2		3.7	100
				4.3	133
	4	2		6.6	136
				7.9	160

Diagram No. 19



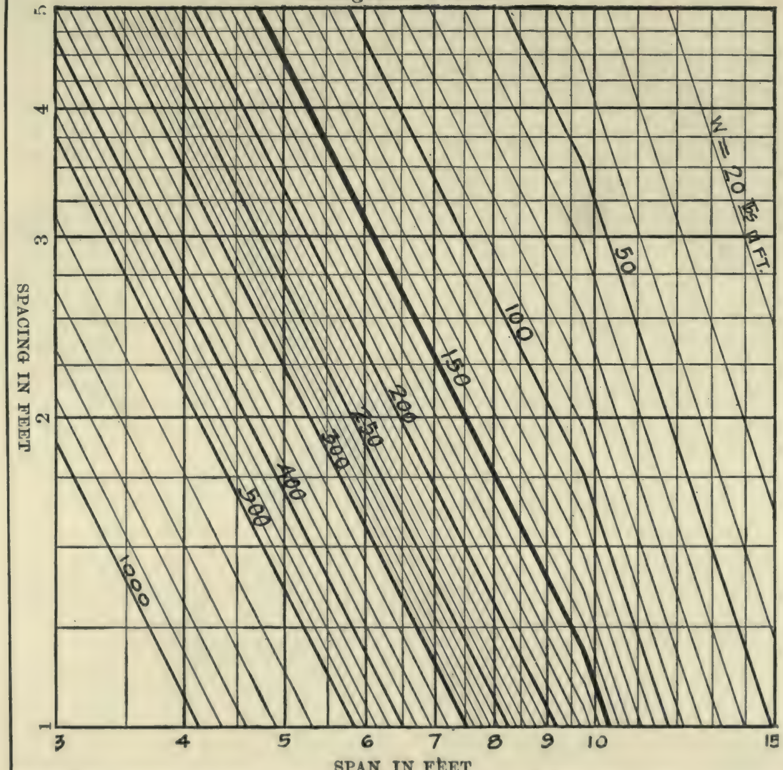
1	2	3	4	5	6
	Ins	Ins	Ins	Lbs.	% W.
Angles	2	2½	3/16	2.8	100
			¼	3.7	130
			5/16	4.5	160
			⅜	5.3	190
	2½	2½	3/16	3.1	103
			¼	4.1	138
			5/16	5.0	165
			⅜	5.9	196
	3	2½	¼	4.5	138
			5/16	5.5	170
			⅜	6.6	200
			7/16	7.6	228
			½	8.5	255
	3½	2½	¼	4.9	140
			5/16	6.1	170
			⅜	7.2	200
			7/16	8.3	230
			½	9.4	260
Tees	2¼	2¼		4.1	110
				4.9	145
	2½	2½		5.5	170
				6.4	200
	3	2½		6.1	180
				7.2	200
	4	2½		7.3	190
				8.6	210
	4½	2½		8.0	190
				9.3	220
	2½	2¾		5.8	200
				6.7	250

Diagram No. 20



1	2	3	4	5	6
	Ins	Ins	Ins	Lbs.	% W.
Angles	2½	3	¼	4.5	100
			5/16	5.5	123
			⅜	6.6	145
			7/16	7.6	166
			½	8.5	185
	3	3	¼	4.9	103
			5/16	6.1	127
			⅜	7.2	148
			7/16	8.3	170
			½	9.3	190
	3½	3	5/16	6.6	129
			⅜	7.8	152
			7/16	9.1	175
			½	10.2	196
	4	3	5/16	7.1	132
			⅜	8.5	155
			7/16	9.8	177
			½	11.1	200
	5	3	5/16	8.2	134
			⅜	9.8	160
			7/16	11.3	180
			½	12.8	206
Tees	2½	3		6.1	136
				7.2	156
				6.6	132
				7.8	153
				9.1	180
				10.0	196
				7.8	128
				8.5	157
				10.9	200
				9.3	157
				8.5	145
				10.0	168
				13.6	210

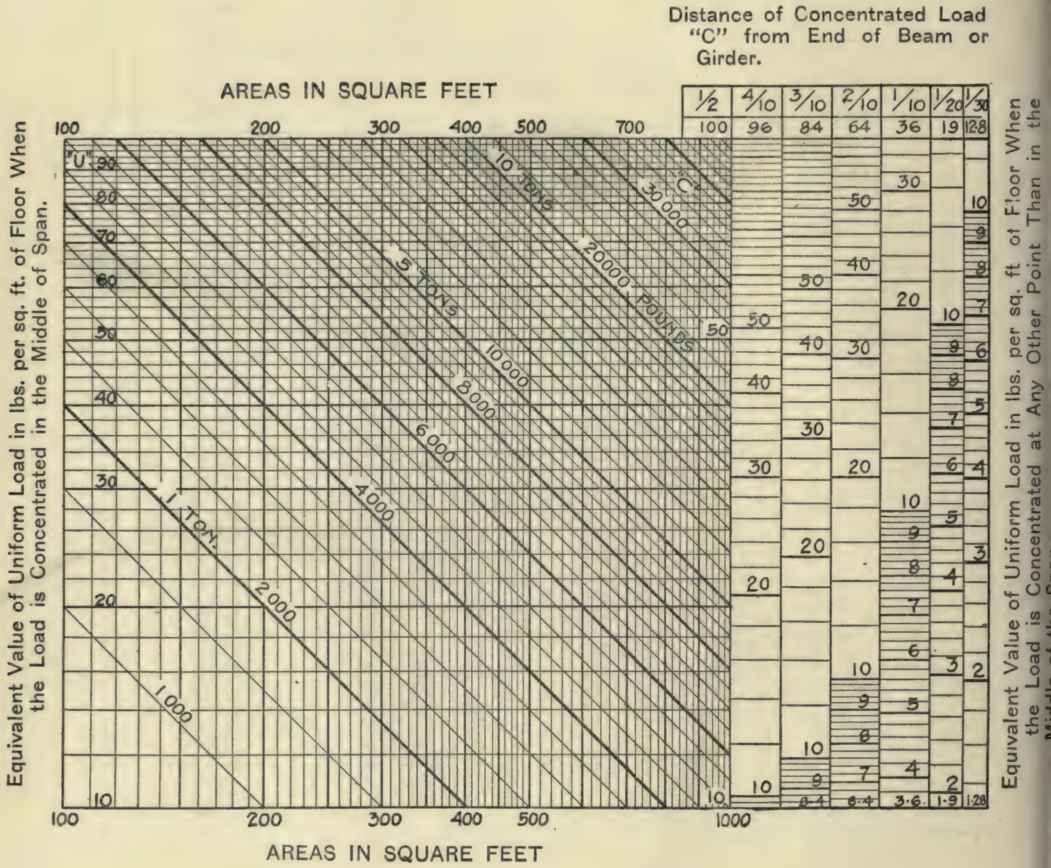
Diagram No. 21



1	2	3	4	5	6
	Ins	Ins	Ins	Lbs.	%. W.
Angles	2½	3½	¼	4.9	42
			5/16	6.1	58
			⅜	7.2	68
			7/16	8.3	79
			½	9.4	88
	3	3½	5/16	6.6	60
			⅜	7.8	71
			7/16	9.1	81
			½	10.2	91
	5	3½	5/16	8.7	64
			⅜	10.4	76
			7/16	12.0	87
			½	13.6	98
	6	3½	⅜	11.7	77
			7/16	13.5	88
			½	15.3	100
	7	3½	7/16	15.0	92
			½	17.0	102
			9/16	19.0	113
			⅝	21.0	124
Tees	4½	3½		15.8	134
	3½	3½		11.7	95
				9.2	75
	3	3½		10.9	93
				9.8	86

Diagram No. 22

For reducing the value of a concentrated load to an equivalent value of uniform load per unit floor area.



Example.—Suppose a post from a stair platform carries 10,000 lbs. to a point 4 ft. from the end of a 20-ft. girder; suppose one-half the sum of the spans of the beams framing into the girder is 20 ft., thus giving 400 ft. tributary floor area upon which the normal live and dead load is 160 lbs. per sq. ft.

The diagram gives 32 lbs. per sq. ft. for a load concentrated $^2/_{10}$ lbs. of the span from the end. Thus it is necessary to design this girder for a uniform load of 192 lbs. per sq. ft.

CHAPTER IV.—SPANDREL BEAMS.

The diagrams and tables given in Chapter III on beams used in floor framing cover the large majority of cases of beamwork arising in building design. Numerous cases arise, however, in connection with building work in which beams are required to carry loads that are not reduced to the same unit load as in floor design. These are various but will be classed, for convenience, under the head of spandrel beams.

Spandrel beams carry a variety of loads ranging from continuous curtain walls and individual piers to extraneous loads from balconies and the like. Two or more of these loads may be combined on the same beam.

A system is herewith presented for the design of such beams. It calls for a minimum expenditure of time and insures a high degree of safety in the solution of this problem. The distinguishing feature of the system is the method of considering a concentrated load for *any position* it may have upon the beam. For a load uniformly distributed or a load concentrated at the middle of a beam, the system differs but slightly from the ordinary methods, as will be seen by referring to the explanation of Diagrams Nos. 24, 25 and 26.

As explained in Chapter I under "Loads on Beams," the effect of a load concentrated at any other point than the middle is less than the effect of the same load if concentrated at the middle of the beam. Evidently therefore, there is always an equivalent reduced load that will have the same effect on a beam when concentrated at the middle as the actual load when concentrated at any other point. This equivalent reduced load is given on Diagram No. 23 and the method of obtaining it will be understood from the following:

EQUIVALENT LOAD DIAGRAM.—In this Diagram No. 23, the abscissas represent the position of the load, and the ordinates represent equivalent load. The actual concentrated load is shown as a curved line, and the position of the load is given in various ways. At the top of the diagram the abscissa scale shows the distance of the load from the end of the beam as a fraction of the

span. At the bottom of the diagram the distance from the load to the end of the beam is given in feet in different scales for various spans from 10 to 26 ft. The ordinate scale gives the equivalent load concentrated at the middle.

To use the diagram, take an abscissa representing the distance of the load from the end of the beam (selecting one of the foot scales for the given span, or else using the fraction scale at the top of the diagram) and follow up to the curved line representing the load. Follow the horizontal at the intersection reading equivalent reduced middle concentrated load.

The diagram also has a horizontal scale showing the reaction at the end nearest to the load, in per cent. of the actual concentrated load. This value of reaction is to be used in investigating the end shear or tendency of the web to buckle by comparing with the maximum allowable end shear on any beam, as given in the tables in Chapter VIII, or as given in the supplementary tables in Chapter III. The subject of end reactions is treated more fully in Chapter VI.

DIAGRAMS FOR SAFE LOADS ON I-BEAMS AND CHANNELS. —The Diagrams Nos. 24, 25 and 26 are for the design of miscellaneous beams and girders conveniently classed under the head of spandrel beams. If time was not an important item in the design of beamwork, these diagrams could take the place of those preceding them, for they are adapted for general application.

The method of using these diagrams will be clear. In each of the three diagrams, the abscissas represent span of beam in feet, while the ordinates represent uniform total load, or total load concentrated at the middle. The diagonal lines represent the different sizes of I-beams and channels. The smaller sizes of I-beams are shown on Diagram No. 24, while the larger sizes, 15-in. and over are shown on Diagram No. 25. All sizes of channels are shown on Diagram No. 26. The heavy full diagonal lines on these diagrams show the "standard" sections of different sizes; the light full line show the special sections. As will be understood, the change in direction of the lines is due to the deflection entering as a limiting factor above a certain span. The reason for the presence of dotted lines parallel to these lines will be given in Note 2.

In these diagrams also, the left hand portion of the lines, that determined by the *allowable* fiber stress, is continued as a dotted line beyond the point where deflection enters as a factor. This dotted continuation can be used when a beam is to be designed without reference to deflection.

To use the diagrams take an abscissa equal to the span, the ordinate (on the proper scale, either for load uniformly distributed or concentrated at the middle) equal to the load, and the section to be used is read off on the diagonal at the intersection. Obviously, the diagram may also be used to give maximum load or maximum span for any given section of I-beam or channel, on a given span or under a given load, respectively.

NOTE 1.—The special 15-in. I-beams run up among the 18-in. and 20-in. I-beams, but to discriminate it is only necessary to note the point of deflection of the lines. The same rule applies to the diagram for channels.

NOTE 2.—If the loads on a beam or girder consist of comparatively light concentrations on top of a well distributed uniform load, it is advisable to reduce the concentrated loads to an equivalent uniform loading. The reason for this will be evident from the following: As the span in inches of a steel beam which is *uniformly loaded* increases above 21.75 times its depth in inches, the maximum fiber stress begins to decrease from 16,000 lbs. per sq. in. if designed for a limiting deflection of one four-hundredth of the span. In the case of a load *concentrated in the middle*, the decrease does not begin to take place until the span is 27.2 times the depth, or 1¼ times that for uniform loading.

On Diagrams Nos. 24, 25 and 26, this difference in safe deflection is shown graphically by dotted lines for beams with uniform loading and by full lines for beams with load concentrated in the middle.

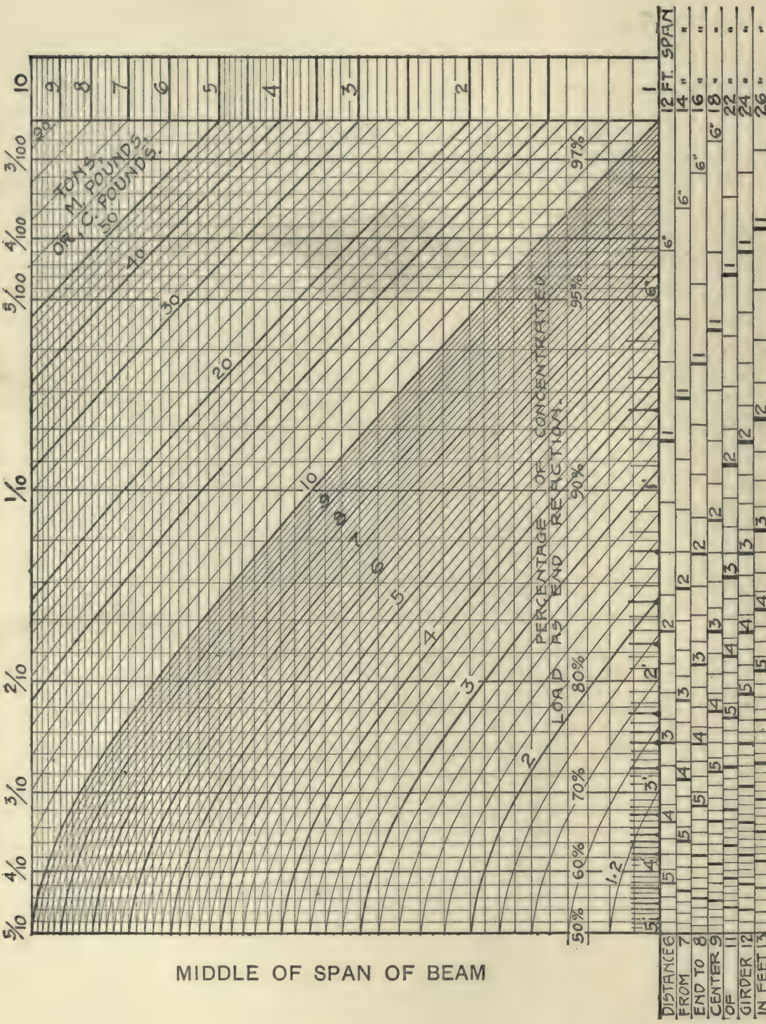
NOTE 3.—When large concentrated loads occur at both ends of a beam the diagrams should not be used. Use tables in Chapter VIII.

Equivalent load in tons, M. pounds or C. pounds.

Diagram No. 23

For Giving an Equivalent Value of Load Concentrated in the Middle of a Span for Any Value of a Concentrated Load at Any Other Point.

Distance of Concentrated Load from End of Beam as a Fraction of the Span

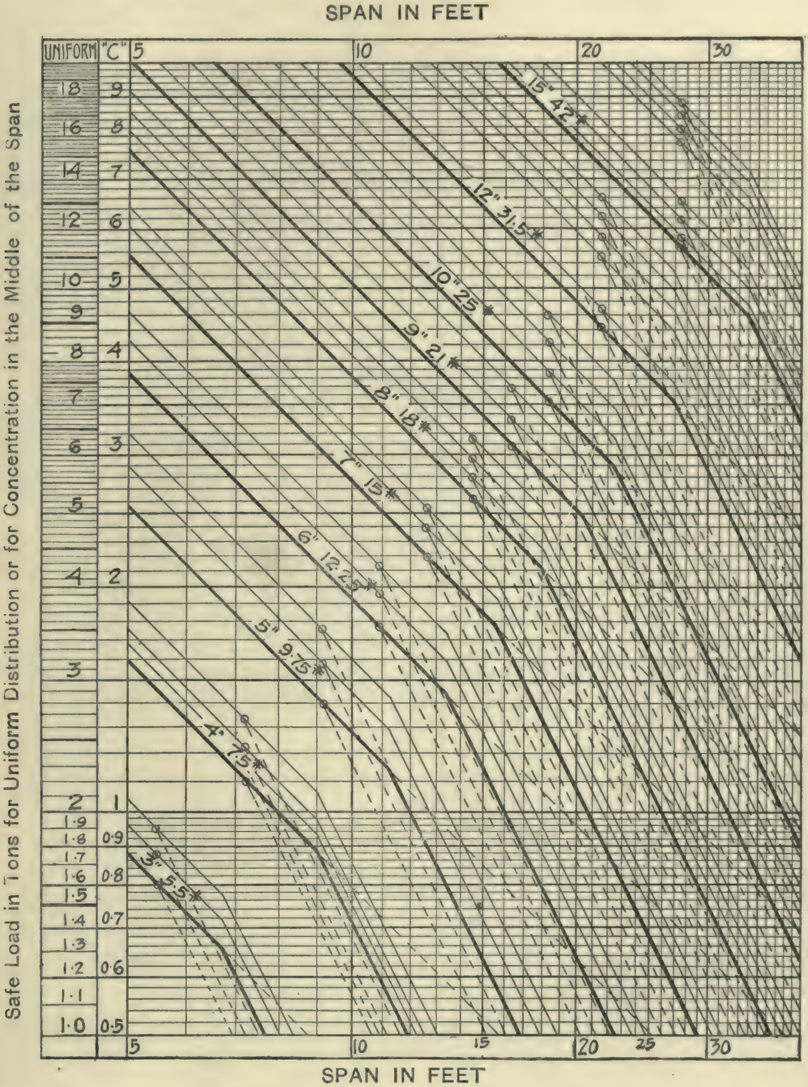


Supplementary Scales for Distance of Concentrated Load from End of Beam in Feet for Various Spans

Example.—A beam supports a load of 1,000 pounds (10 C. or 1 M.) 1 ft. from the end of a 12-ft. span. What equivalent value of load concentrated in the middle would require a beam of the same strength? Answer: 305 pounds (3.05 C. or 0.305 M.).

Diagram No. 24.

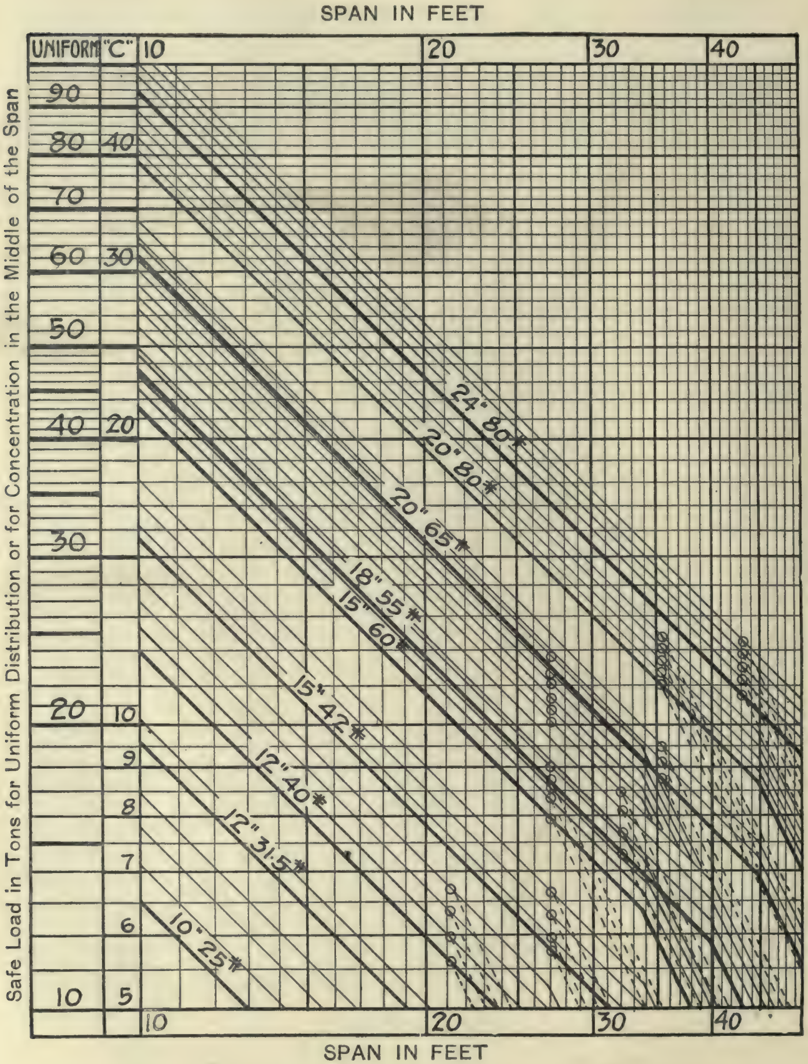
For giving the allowable load on standard and special I-beams.



The deflection curves are shown by full lines for loading concentrated in the middle and dotted lines for uniformly distributed; thus a 6-in. × 12¼-lb. beam with a span of 15 ft. will carry 1.15 tons concentrated in the middle or 1.85 tons uniformly distributed.

Diagram No. 25

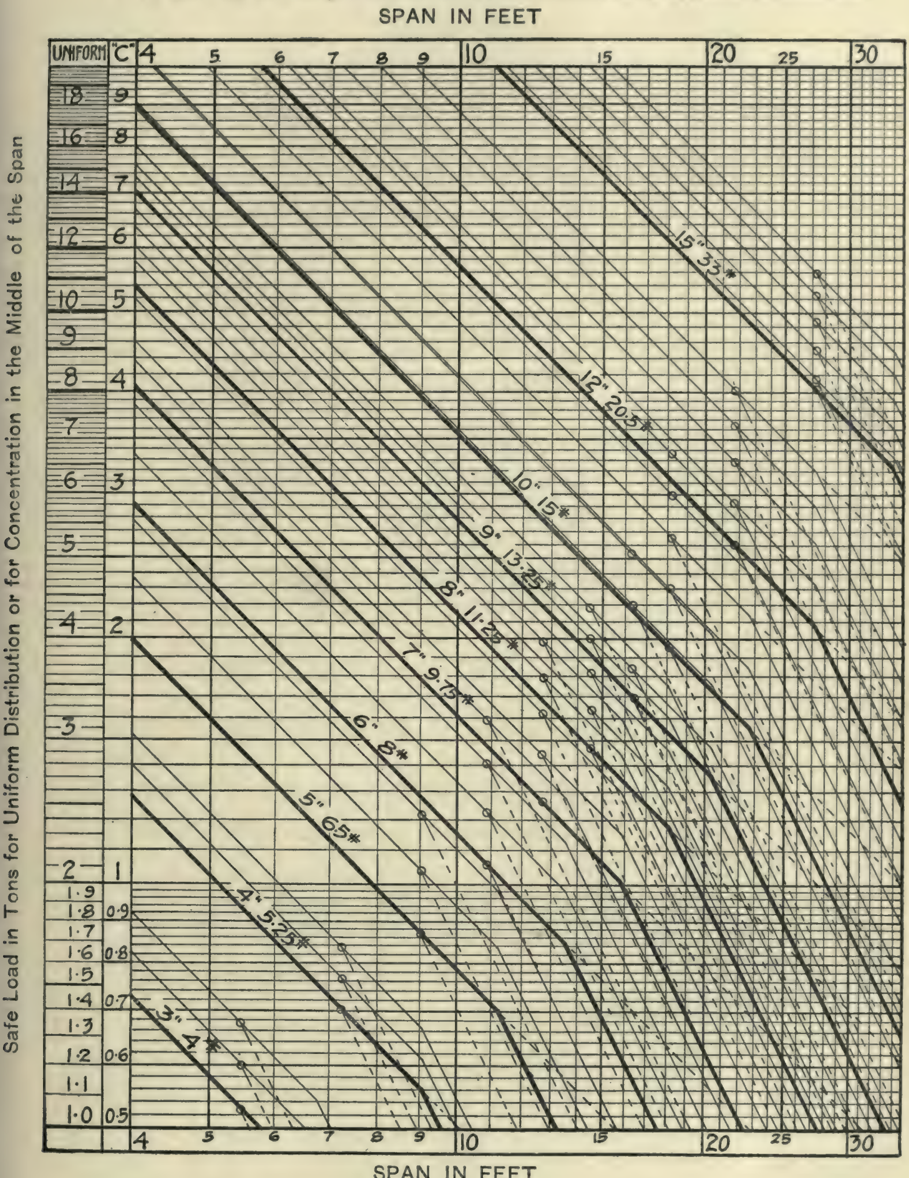
For giving the allowable load on standard and special I-beams.



Example.—A 20-in. × 65-lb. beam 42-ft. span will carry 7.35 tons concentrated in the middle of the span or 12.35 tons uniformly distributed.

Diagram No. 26

For giving the allowable load on standard and special channels.



Example.—A 12-in. × 20.5-lb. beam 24-ft. span will carry 2.35 tons concentrated in the middle of the span or 4.2 tons uniformly distributed.

CHAPTER V.—GRILLAGE BEAMS.

FOOTINGS.—The use of steel beams in footings is a problem of frequent occurrence with the structural designer. This chapter deals with the design of the steel beams for grillage footings and presupposes a general acquaintance with the subject of foundations* on the part of the reader.

Grillage footings are in general either footings of walls or footings of columns. The former are somewhat the simpler in design, but their treatment is precisely the same as that for column footings. They will therefore not be especially referred to in the following. When two or more columns occur on a single footing the problem, on the other hand, becomes so complex that only those familiar with the mechanics of engineering should presume to deal with it. The properties of beams as given in Chapter VIII may be used for this latter case.

The known quantities in the design of a footing are the load on the column and the allowable pressure per unit area on the soil (or other material) which supports the grillage. The area of the surface of a footing which comes in contact with the soil is found by dividing the load on the footing by the allowable pressure on the soil.

The accompanying drawing, Fig. 2, shows a design of a grillage footing for a single column having two tiers of grillage. From this drawing the elements of the design of the different tiers of beams may readily be understood. The load on the column is transferred to the grillage by a "base," of cast iron or of steel, and the size of this base must be known or assumed before the grillage beams can be designed.

DESIGN OF GRILLAGE BEAMS.

BENDING.—As explained in the short treatment on the "Conventional Methods of Considering Loads on Grillage Beams" in Chapter I there are two methods of designing these beams.

The *most common method* is to consider only the projecting length of these beams beyond the edge of the tier or base imme-

*For this aspect of the subject such books as Baker's Treatise on Masonry Construction; Kidder's Building Construction and Superintendence, Part I; and Patton's Practical Treatise on Foundations, and the back files of the Engineering News and Engineering Record should be consulted.

diately above them. Within certain limits this method will give satisfactory results. It will always give safe results if $^b/_{2a}$ does not exceed 0.3, where b equals the breadth of the base or tier above, and a equals the projection of the beams beyond the edge of this base—$(2a + b)$ equals the entire length of the grillage beam under consideration.

The *second method* is more conservative although the following is a modification of it by 30% limit allowance for the value

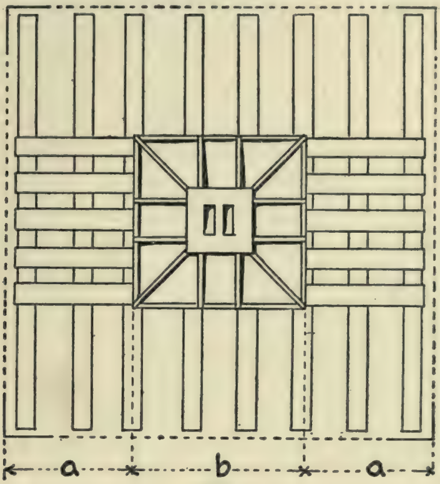


Fig. 2.

of the concrete in the footing. It should be stated at the outset that Diagram No. 27 is to be used for both methods.

When $^b/_{2a}$ exceeds 0.3 it is generally advisable to increase the number of beams found by the first method. This increase can be made in two ways:

(1) If d is the distance center to center of grillage beams, as found on Diagram No. 27 (see explanations following), a and b same as before, and if c equals the new distance c. to c., then

$$c = \frac{1.3\,a}{a + {}^b/_2}\,d. \tag{20}$$

(2) If n equals the number of beams in the tier (found by dividing the total width of the footing by d, as given on the diagram by the first method) and x equals the number required by the second method, then

$$x = \frac{7\,a + 5\,b}{10\,a}\,n. \tag{21}$$

These formulas will be more fully illustrated in the example on page 64.

While the bending is the main item that determines the strength of grillage beams, one other determinative factor should not be neglected:

BUCKLING OF THE WEB.—The following is practically an empirical method of ensuring the safety of the webs of grillage beams against buckling. Here again the concrete is undoubtedly a factor in the strength of the webs, but as it is probably very slight, its influence is not considered. Tables in Chapter VIII give values in tons per lineal foot of I-beams for the allowable load per lineal foot that can be placed on such a beam. These values were computed according to the "Code" on the basis of the assumption that the safe load on the flange of one lineal foot of beam could be taken as a safe unit working load per foot for the total length of beams directly under the base.

NOTE.—The flanges of nearly all rolled beams are usually a little higher on one side than on the other, thus there is a slight tendency for eccentricity of loading on the web. It is therefore advisable to be liberal in the use of separators between grillage beams, especially directly under the edges of the tier above.

DIAGRAM FOR GRILLAGE BEAMS.—The make up of Diagram No. 27 is quite simple. Abscissas represent span; i. e., projecting portion of beams (symbol "a") and ordinates represent the safe loads; diagonal lines represent the different sizes and weights of beams. The safe loads are expressed in tons per square foot of area, and therefore different ordinate scales are necessary for different spacings of beams. These suitable scales are shown at the left and right of the diagram, so that 8, 10, 12, 14 or 16-in. spacing of beams are represented—the scale given on the diagram proper is for 12-in. spacing, while the other scales are found at the sides.

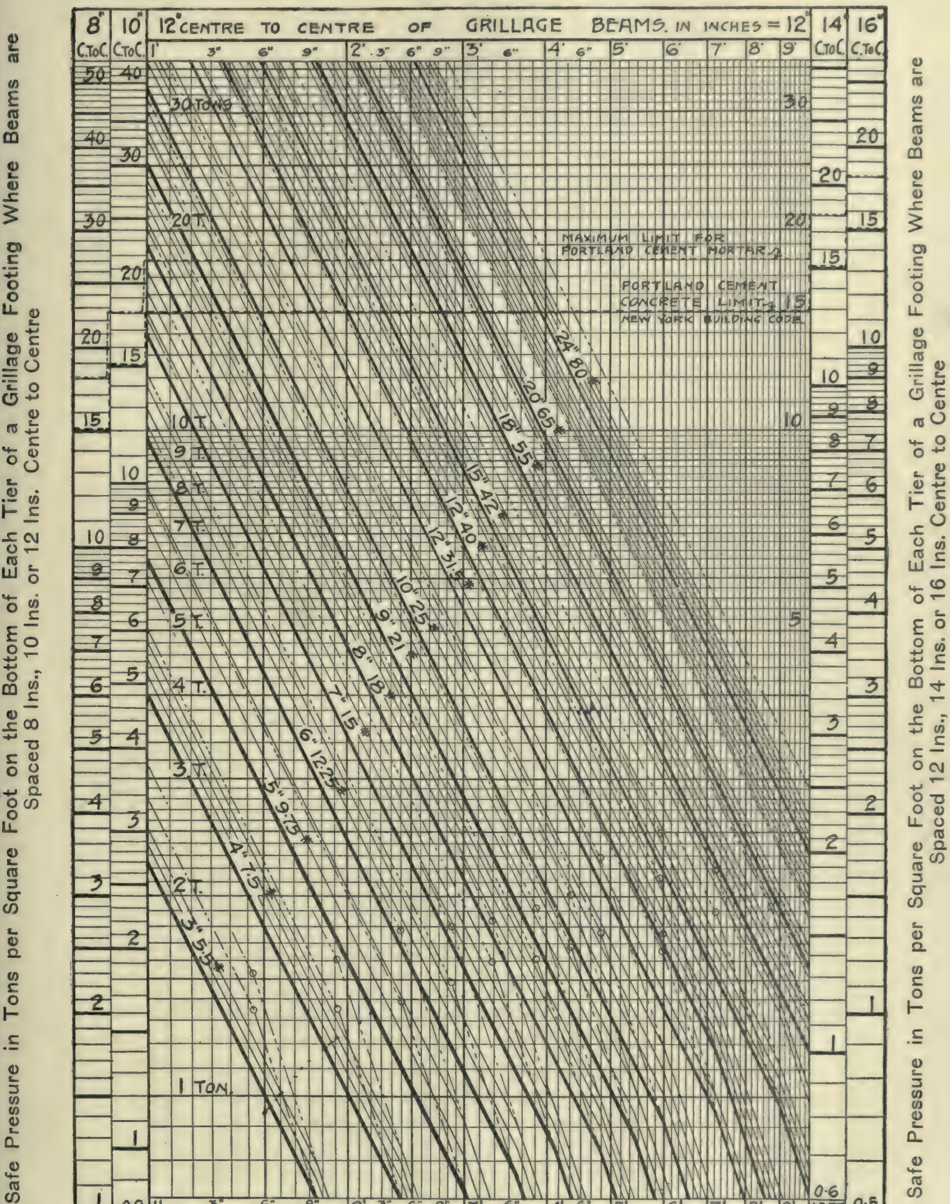
NOTE 1.—The loads on grillage beams vary inversely as the spacing of the beams, for instance, suppose 20-in. spacing was desired, evidently half the load given in the diagram for 10-in. spacing will give the correct size of beam for 20-in. spacing.

NOTE 2.—The full heavy lines on the diagram are for the standard minimum weight I-beams, and the fine lines for other weights. These lines give values on a basis of 16,000 lbs. per sq. in. max. fiber stress. The dotted lines produced from the right hand portions of the curves, at the bend of the curves, give values for the standard minimum weight I-beams on a basis of a fiber stress of 20,000 lbs. per sq. in. and a limiting deflection of one five-hundredth of the span.

NOTE 3.—The limiting values for the pressure on mortar and concrete are indicated by dotted lines across the diagram. It is advisable to keep well below these limits.

Diagram No. 27

For giving size, weight and spacing of I-beams necessary for each of the several tiers in a grillage footing.



PROJECTION OF GRILLAGE BEAMS IN FEET

Example.—Grillage beams 10 ins. × 25 lbs. projecting 3 ft. and spaced 8 ins. c. to c. are good for 5.4 tons per square foot; 10 ins. c. to c., 4.3 tons; 12 ins., 3.6 tons; 14 ins., 3.1 tons, and 16 ins., 2.7 tons. (See text.)

To use the diagram for the design of grillage beams, take an abscissa to represent the projection of the beams in feet, an ordinate equal to the load per square foot, using either the ordinate scale in the diagram proper or the supplementary scales, and follow to an intersection. The diagonal line nearest above this intersection gives the size and weight of beam.

In preliminary work several spacings of beams should be assumed and the resulting size of beam obtained for the conditions of load and span already determined. A rough calculation of the weights of the beams will then indicate the most economical for the design.

Example.—Suppose a grillage footing is to be designed for a column carrying a total load of 400 tons, where the soil will bear a safe loading of 4 tons per sq. ft.; suppose the cast-iron base of the column is 4 ft. square. The bottom tier would then be 10 × 10 ft. (100 sq. ft. in area), and the next tier 4 × 10 ft. (40 sq. ft.), which would give 10 tons per sq. ft. pressure upon it. The projection of the beams will be 3 ft. in both cases.

Solution by 1st Method.—Using Diagram No. 27. The beams required for a span of 3 ft. and 4 ton loading may be 10-in. 25-lb. Is, spaced about 10-in. c. to c., giving 12 beams to the tier, or 12-in. 31½ lb. Is spaced about 16-in. c. to c., giving 8 beams to the tier. Thus 2,560 lbs. of steel is needed if 12-in. beams are used, and 3,000 lbs. if the 10-in. are used.

For the next tier, 15-in. 42-lb. Is can be spaced 10-in. c. to c., i. e., 5 beams are required (2,100 lbs.), or 18-in. 55-lb. Is, spaced 16-in. c. to c., i. e., 3 beams (2,200 lbs.) to the tier.

Solution by 2d Method.—(A) Using formula (20). The beams should only be spaced 78% of that given by the diagram. Thus, the 10-in Is should be 7.8-in. c. to c., the 12-in. Is 12.5-in. c. to c.; for the next tier the 15-in. Is 7.8-in. c. to c. and the 18-in. Is 12.5-in. c. to c.

(B) Or using formula (21) the number of beams should be 137% of those given by the first method. That is, instead of 12 and 8 beams for the first tier, there should be 16 and 11, and for the next tier, instead of 5 and 3 there should be 7 and 4 beams.

Solution for Buckling of the Webs.—The tier of beams directly under the base of the column always presents the most unfavorable conditions. This cast-iron base is 4 ft. square. By dividing the load (400 tons) on the footing by the length of the base, and the quotient by the number of beams in the tier, the load on each lineal foot of beam is obtained. Thus, on the 15-in. beams (considering the first method of design), this load is 20 tons, and for the 18-in. beams the load is 33.3 tons. By referring to the tables in Chapter VIII it will be found that the allowable load on the 12-in. beams is 22.3 tons, while the 18-in. beams only carry 23.4 tons. In the latter case it is evident that it would be impracticable to use these three beams.

CHAPTER VI.—END REACTIONS.

The effect of the end reaction on the strength of a beam has been discussed in Chapter I; the cause and effect of an end reaction was referred to in Chapters III and IV; while this chapter briefly outlines the subject of end reactions as considered by the structural designer in deciding upon the relative value of standard and special details of construction, and in deciding upon the design of these standard or special details.

Standard details materially reduce the cost of structural work, and so long as there is a balance of saving in shop work to pay for the excess material required by their use, they should be adopted. On the other hand, the most liberal standards are not safe beyond partially defined limits and these limits should be carefully checked. It is hoped this treatment of the subject will be of value in keeping a proper balance between these conflicting conditions.

From the following description it will be seen that Diagram No. 28, gives the end reaction, due to uniformly loading a beam of any span so as to develop its full working strength. It also gives equivalent values for area of a bearing plate or number of rivets required for a pair of connection angles.

DIAGRAM NO. 28.—This diagram covers cases of end reactions on beams under uniform loads. The abscissas represent span of beam, ordinates represent end reaction and diagonal lines represent the different sizes of I-beams and channels. The quantities referred to are all found on the central part, or diagram proper. To the left and right will be found alternative sets of ordinate scales, applying directly to the diagram which give the essential values concerning riveted end connections and bearing plates. The six scales to the left of the diagram show the number of ¾-in. rivets, in shear, required to take care of a reaction of any given amount. Steel and wrought iron shop rivets, field rivets and bolts are represented by the six columns. The two columns at the right of the diagram should be used in connection with these; they show the number of ¾-in rivets, in bearing on a web 1/10-in. thick, required for any reaction. These values are also given for steel and wrought iron rivets. At the right of the latter scales is another set of ordinate scales applying to bearing plates (often called templates). They show the area (in square inches) of bearing plate

required for any reaction, in five columns, calculated on a basis of 18 tons, 15 tons, 5 tons and 8 tons permissible loadings per square foot. The usual sizes of bearing plates are given in the last column to the right, at proper intervals to represent their area in square inches.

To use the diagram and auxiliary scales for any particular problem, take an abscissa representing the span in feet, and follow along the vertical to the intersection with the diagonal representing the size of I-beam or channel used. The horizontal at the intersection, if followed to the reaction scale at the left of the diagram, gives the actual reaction in tons. Following the same horizontal farther to the left, the number of rivets (or bolts) required in shear is found in the appropriate column. The same horizontal followed out to the right shows the number of rivets required for bearing on 1/10 in. of web. This number is to be divided by the actual thickness in tenths of an inch of the metal, and the quotient then represents the number of rivets required for bearing.

In case the end of the beam rests on a bearing plate, the horizontal line is followed out to the scales "Sizes of Templates," and on the appropriate scale (as described below) the required area in square inches is read off.

NOTE.—For spans other than those given on the diagrams the reaction is given by the following rule: For any given size of beam, the end reaction varies inversely as the span.

Example.—The end reaction on a 10-in. 25-lb. I-beam, 10-ft. span, loaded to its full capacity, is 6½ tons. If the span were 30 ft. the reaction would be one-third of 6½, or 2.2 tons.

The values to be obtained from the diagram just described are used in the design of connection angles and bearing plates, along with the following table and diagram.

DESIGN OF CONNECTION ANGLES.

STANDARD CONNECTION ANGLES:—The sizes of angles used for "standards" are closely represented by the following list:

	For 18" Is and over.	For 15" Is and channels and under.
Passaic, Carnegie, Cambria, Jones & L.	4 × 4 × 3/8	6 × 4 × 3/8
Pencoyd	4 × 3½ × 7/16	6 × 3½ × 7/16
Am. Bridge	4 × 4 × 7/16	6 × 4 × 7/16
Anonymous	3½ × 3½ × 3/8	3½ × 3½ × 5/16

The lengths of standard angles for the several sizes of beams are usually as follows:

For	.....24" Is	1' 6"	l g.	For	..10" to 7" Is and [s	0' 5"	l g.
"	.....20" and 18"	1' 3"	"	"	.. 6"	0' 3"	"
"	.....15" Is and [s	0' 10"	"	"	.. 5"	0' 2½"	"
"	.....12" Is and [s	0' 7½"	"	"	.. 4" and 3"	0' 2"	"

The numbers of shop and field rivets in the above standards do not vary very much, although Pencoyd and the American Bridge Co. put in one extra shop rivet in the 15-in. and in the 10-in. to 7-in. standards over what is called for in the same standards in the following table.

The strength of a pair of connection angles (sometimes called knees) not only depends upon the number and strength of the rivets in them, but also upon the strength of the metal upon which the rivets bear. These values for both standard and special connection angles are given in the following table.

TABLE GIVING MAXIMUM ALLOWABLE END REACTION ON STANDARD AND SPECIAL CONNECTION ANGLES:—This table (No. 16) gives the maximum load that can be safely carried by different standard and special connection angles in terms of the strength of the shop and field rivets in shear or bearing. Evidently, the end reaction on the beam must not be greater than these values for shear or for bearing. The tabular values are for safe shearing capacity (single shear) and safe bearing capacity (for bearing on $^1/_{10}$-in. metal in web). The construction of the table needs no further explanation. Whenever there is reason to suspect that a standard connection will not be sufficient for conditions indicated by Diagram No. 28, the capacity of this standard should be found from the table. The use of this table is best illustrated by an example.

Example:—A 12-in. 31.5 lb. I-beam has 10-in. beams 16½ ft. long framing into it from both sides. The web of the 12-in. girder is 0.35 in. thick, thus giving a bearing value of 0.175-in. for each field connection. Suppose, wrought iron rivets are used in the field, the permissible end reaction would then be 0.175 × 2.26, or 3.95 tons. According to Diagram No. 28 this is the safe limit for uniformly loading the 10-in. beams.

In designing special connection angles their length is limited by the clear distance between the fillets of the beam. This distance is given for all I-beams and channels in the tables given in Chapter VIII.

On general principles it is advisable to use as large connection

angles as practicable, because they add stiffness to the framing. This is especially of value in high buildings.

DESIGN OF BEARING PLATES.

The elements in the design of a bearing plate or template are its bearing area and its thickness.

The *bearing area* depends upon the load to be carried (the end reaction) and upon the permissible unit load on the material supporting the template. This latter is specified in the "Code" (N. Y. C.) as follows:

On Brickwork:
- 8 tons per sq. ft. when lime mortar is used;
- 15 tons per sq. ft. when cement mortar is used;
- 18 tons per sq. ft. when Portland cement (1 to 3) is used.

For Rubble Masonry:
- 5 tons per sq. ft. when lime mortar is used.

These values will be found to be the column headings for templates in Diagram No. 28; then, by the use of this diagram the area of bearing required is at once obtained, and a convenient size of template selected.

The *thickness of the bearing plate* depends upon the material of the plate, the amount of projection of the plate beyond the flange of the beam or beams supported,* and the unit pressure on the bottom of the plate. The material may be cast iron, wrought iron or steel; the unit pressure on the bottom of the plate is that used in finding the area of the plate (i. e., 5, 8, 15, or 18 tons per sq. ft., as above).

DIAGRAM NO. 29 enables the required thickness to be readily obtained. The abscissas are amount of projection in inches. The ordinates are thickness of plate, there being three different ordinate scales, for cast iron, wrought iron and steel, respectively. Different diagonal lines represent the different allowable unit pressures on the supporting material.

To use the diagram, take an abscissa equal to the projection of the plate, in inches, and follow up to the diagonal. The horizontal at the intersection gives, when followed to the right, the thickness of the plate.

---

*The clear distance between the inside flanges of a pair of beams must not exceed 2.45 times the projection of the plate beyond the outside flanges.

Table Giving Maximum Allowable End Reaction on Standard and Special Connection Angles

TABLE 16 — All Rivets ¾″ Diam.

Number of Holes in Shop or Field End of Connection Angles.		SHEARING — Values Given for Single Shear						BEARING — Values Given for $\frac{7}{16}$-inch Bearing of Rivets			
		Steel			Wrought Iron			Steel		Wrought Iron	
		Shop Rivet	Field		Shop Rivet	Field					
Shop	Field		Rivet	Bolt		Rivet	Bolt	Shop	Field	Shop	Field
		Tons	Tons	Tons	Tons	Tons	Tons	T'ns	Tons	Tons	Tons
a2	2	4.4	3.5	3.1	3.3	2.6	2.4	1.5	1.5	1.13	1.13
b3	4	6.6	7.0	6.2	5.0	5.2	4.8	2.2	3.0	1.70	2.26
c5	6	11.0	10.5	9.3	8.3	7.8	7.2	3.7	4.5	2.82	3.39
d5	8	11.0	14.0	12.4	8.3	10.4	9.6	3.7	6.0	2.82	4.52
e5	10	11.0	17.5	15.5	8.3	13.0	12.0	3.7	7.5	2.82	5.65
f6	12	13.2	21.0	18.6	9.9	15.6	14.4	4.5	9.0	3.39	6.78
7	14	15.4	24.5	21.7	11.5	18.2	16.8	5.2	10.5	3.96	7.91
8	16	17.6	28.0	24.8	13.2	20.8	19.2	6.0	12.0	4.52	9.04
9	18	19.8	31.5	27.9	14.6	23.4	21.6	6.7	13.5	5.08	10.17
10	20	22.0	35.0	31.0	16.5	26.0	24.0	7.5	15.0	5.65	11.30

Relative Values of the Several Sizes of Rivets

Sizes of Rivets	Ratio in Shear	Bearing—Ratio
⅜″ rivet is to ¾″	as 1 is to 4	or as 3 is to 6
½ " " " ¾	" 4 " " 9	" " 4 " " 6
⅝ " " " ¾	" 11 " " 16	" " 5 " " 6
⅞ " " " ¾	" 4 " " 3	" " 7 " " 6
1 " " " ¾	" 9 " " 5	" " 8 " " 6
1⅛ " " " ¾	" 9 " " 4	" " 9 " " 6

a is for standard connections for 3″ to 6″ I, and [,
b " " " " " 7 to 10 " " "
c " " " " " 12 " " "
d " " " " " 15 " " "
e " " " " " 18 and 20 "
f " " " " " 24 "

Diagram No. 28

For giving values for rivet requirements in connection angles, also areas for bearing plates.

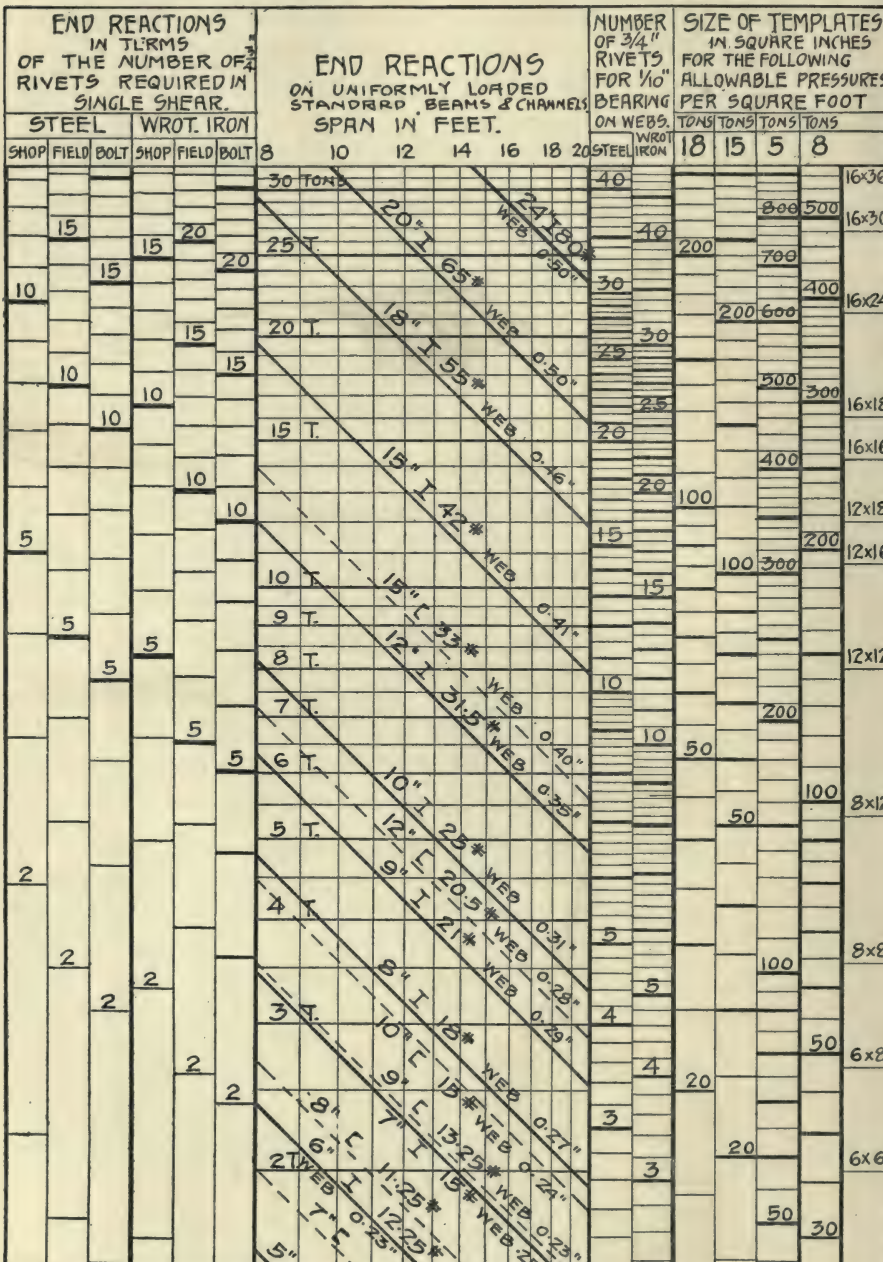
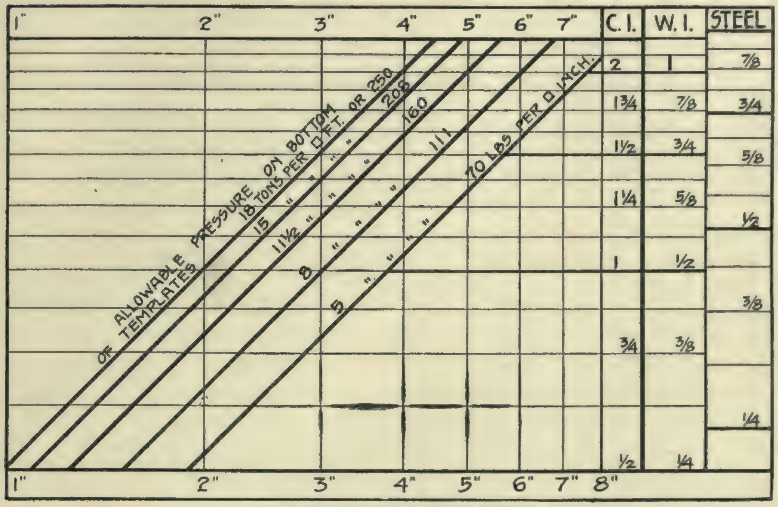


Diagram No. 29.

For giving thickness of bearing plates of cast iron, wrought iron or steel.



Note.—When the clear distance between the inside flanges of a pair of beams supported by a single plate exceeds 2.45 times the projection of the plate beyond the outside flanges, the thickness of the plate is to be obtained by the following modification of abscissa value for projection (not the real projection). Multiply the clear inside distance between flanges by 0.41 and use this value on the abscissa scale.

Part III.—Columns and Truss Members.

CHAPTER VII.—STEEL COLUMNS.

The usual methods of designing built steel columns are either quite complex or else they apply to specific forms of section, as for instance, to the so-called zee-bar, channel or plate and angle columns. In the following system of diagrams and tables, however, the form and make up of the section is treated as a subject for secondary rather than primary consideration.

The radius of gyration is an important factor in the design of columns. Numerically the radius of gyration is the square root of the quotient obtained by dividing the moment of inertia of a cross section by the area of the section. This mathematical computation is quite laborious in the case of a built up section, mainly because it has to be repeated for every variation in area and distribution of the material in the cross section. It is believed, however, that such computations as these are unnecessary. Diagrams Nos. 30 and 31 give the radii of gyration for the different forms of rolled sections and for the most common forms of built sections (see explanations later). A little study of these diagrams will convince the designer that the radius obtained by this graphical means will give results as accurate as the most conservative could wish to use in his computations.

This radius of gyration, r, is one of two variables that determine the ratio of slenderness* of a column. The other is the unsupported length, l, of the column. This ratio of slenderness is expressed by the quotient of $\frac{l}{r}$, which is the factor that determines the allowable unit compressive stress to be obtained from the aforementioned column formulas (see Chapter II). However, in the treatment of the subject of columns herewith, the allowable unit stress does not come directly into question, because it has been taken into account in the construction of the diagrams for safe loads.

*See Chapter II.

THREE STEPS ARE TAKEN IN THE DESIGN OF A COLUMN: First, determination of the ratio of slenderness; second, the area of the cross section is found; third, the make up of the section is decided. These steps will be considered in this order, the third step being taken up in Chapter VIII.

RATIO OF SLENDERNESS.

Three diagrams are given, two of which Nos. 30 and 31 are used for obtaining the radius of gyration for standard sections and for built up sections respectively, and the third—Diagram No. 32—is used for obtaining the ratio of the length of the column to the radius of gyration or the so-called ratio of slenderness of the column.

DIAGRAM NO. 30:—Here abscissas represent thickness of metal expressed as a percentage of the depth or diameter of the section. Ordinates represent the radius of gyration in inches. The curves represent the different arrangements of material as indicated in the left hand margin.

Thus the lowest line is for an H section with neutral axis coincident with the web, or for a star shaped section (it will be observed that the former has a depth greater than the width of the flanges, thus corresponding to the average I-beam properties); the next line is for a *square* H section with neutral axis as before; the next is for a solid circular cross section, etc. The uppermost curve in the diagram (that represented by a full black line) gives the theoretical values for a section of two plates at a fixed distance back to back and gradually reduced in thickness. In practice this section is very nearly represented by two channels or two I-beams latticed when the diameter is taken as the distance apart of their centers of gravity. Theoretically, this distribution of material gives the highest possible radius, but practically, it is impossible to make use of it because the latticing does not transmit the stresses due to eccentric loading satisfactorily.

Beside the diagram proper, supplementary abscissa scales and ordinate scales are given. Since the abscissas represent the thickness of metal in the section expressed as a percentage of the diameter or depth of the section, different scales may be used to give the thickness directly in inches for different diameters. Such a series of scales is given in tabular form above the diagram, covering diameters from 3 ins. to 28 ins.

The principal ordinate scale on the right hand edge of the diagram proper gives the radius of gyration. A series of supplementary scales to the right of this give the radius directly in inches for

any diameter of section. Seven scales are given, covering diameters from 6 ins. to 18 ins. inclusive.

The abscissa scales (those in tabular form above the diagram) show a dotted irregular line which divides the scales into two parts. This line indicates the thickness of metal which the "Code" (N. Y. C.) specifies as the minimum for the different diameters of cast iron columns: Steel column sections are as a general rule found to the left of the line; at any rate they should for the sake of economy fall to the left of the line, since otherwise the metal is needlessly thick and may be better utilized by making the section larger and using thinner metal.

The values in this diagram are entirely independent of the material of the column, and the sections shown cover the common forms of cast iron and wooden columns as well as the built-up sections of steel columns. It should be remembered that for all materials a similar distribution gives similar values for the radius of gyration.

To use Diagram No. 30, after the general style of the section of the column has been decided upon, take an abscissa representing the thickness of metal (either as a percentage of the diameter on the lower scale, or in inches on the proper scale above the diagram) and follow the vertical up to the curve which represents the style of section selected. The horizontal at the intersection gives the radius of gyration on one of the ordinate scales at the right of the diagram.

DIAGRAM NO. 31 is for the same purpose as the preceding except that the sections are for the most common built-up steel columns. Its construction is similar to that of the preceding diagram. Curves are drawn for different sections, and the ordinates to these curves give the radius of gyration in inches; a series of ordinate scales is provided at the right for different values of the diameter (depth) of the section, and the radius of gyration is read off on the appropriate scale. The left hand end of the curves denotes the minimum thickness of metal ordinarily used in practice for any one of the sections represented by a curve, while the right hand end corresponds to the usual maximum thickness employed for that type of section. The curves then show the variation of the radius of gyration as the thickness of metal used increases from the least to the greatest thickness ordinarily used.

NOTE:—Attention is drawn to the fact that in the outline sketches for the different sections represented by curves on this diagram, dimension arrows are shown which indicate the particular dimension taken to be the

"diameter." Thus, in the case of the Z-bar column, the width of the web plate is the diameter for the neutral axis perpendicular to the web.

The use of Diagram No. 31 will be quite clear from the preceding. It presupposes a selection of the type of section to be used in the particular case, and further the ability to judge approximately what relative thickness of metal will be required with that section to carry the load.

DIAGRAM NO. 32:—The *ratio of slenderness* of a column is graphically represented on this diagram. Abscissas represent length in feet and ordinates represent the ratio of slenderness. Curves drawn on the diagram represent various values of the radius of gyration in inches.

It is to be noted that while for convenience the abscissa scale shows length in feet and the radius of gyration in inches the ratio of slenderness—or quotient of the two—is given as if both were in inches.

To use the diagram for any particular problem, take an abscissa equal to the length of the column in feet and follow the vertical up to the curve representing the radius of gyration of the column section selected. The horizontal at the intersection shows on the ordinate scale the desired quotient, i. e., the ratio of the length to the radius of gyration.

SECTION AREAS.

As previously stated the second set of diagrams give the areas of cross sections of columns for known loads and known values for the ratio of slenderness. These areas may conveniently be divided into three classes; area necessary for concentric load when ends of columns are "flat"; area or areas necessary for eccentric load when ends are flat and securely braced laterally in the direction or directions of the eccentricity; and, area necessary when pin ends are used. Values for these three classes of loading are given on Diagrams Nos. 33 and 34—Diagram No. 33 which gives the area required for concentric loading on a column with flat ends being the principal diagram of this group, because the two latter values are given in per cent. of the first. This is more fully explained below under the separate heads.

SECTION AREAS FOR CONCENTRIC LOADING:—Diagram No. 33 gives the safe load on medium steel columns for any ratio of slenderness and any area of cross section. In this diagram,

abscissas represent values for any ratio of slenderness from 10 to 200; ordinates represent the area of cross section in square inches; curves on the diagram represent different values for the safe load capacity of the column.

Several supplementary scales appear on the diagram. At the bottom, just above the main abscissa scale, is a scale showing "Rate of increase of area" for any given load and length of column with varying ratio of slenderness.

This scale will be found valuable in indicating the effect of changes in dimensions of cross sections upon the amount of metal required in a column. Thus it shows the importance of keeping the ratio of slenderness down as low as possible. For instance, a ratio of 170 calls for three times as much material as a ratio of 10.

At the extreme right of the diagram is a scale of weights; it shows the weight of the column per lineal foot for any area of cross section in square inches.

A further supplementary abscissa scale on the lower part of the diagram gives the area for pin end columns in the form of percentage factors, based on the section area required for an equivalent load on a column with flat ends i. e. as found on the main part of the diagram.

SECTION AREAS FOR CONCENTRICALLY LOADED COLUMNS WITH PIN ENDS:—When the ends of a column are hinged or have pin bearings it is less rigid than when the ends are flat. For this reason lower unit stresses are allowed for pin end columns. The "Code" (N. Y. C.) makes an equivalent provision by saying "the working stress in struts of pin connected trusses shall not exceed 75% of the working stresses for flat ends." This gives a flat increase of $33^1/_3$% in the area of the section for all column ratios.

The percentages given by the aforementioned supplementary scale show values increasing as the column ratio increases. Thus, for a column ration of 100 the percentage is 120, i. e., the pin end column should be given one-fifth more section area than a flat-end column with the same ratio of slenderness and with the same load; for a column ratio of 150 the percentage is nearly 155.

SECTION AREAS FOR TENSION MEMBERS:—The scale on Diagram No. 33 to the left of the scale of weights, on the right hand edge of the diagram, gives values for loads in tension for different areas of section (or weights per lineal foot), based on a tensile strength of 16,000 lbs. per sq. in. of net section.

NOTE:—This latter scale has absolutely no bearing on the use of this diagram for the design of columns. It is intended for use in the design of trusses where both compression and tension members occur together and will be very useful for such work. The tables of properties of shapes in Chapter VIII, are convenient for use with this scale as well as for use with the diagram proper.

THE USE OF DIAGRAM NO. 33 will be evident from the preceding. Taking an abscissa equal to the ratio of slenderness of the column, the intersection of the vertical with the curve representing the load gives the required area of section on the ordinate scale to the left, or, the required weight per lineal foot on the ordinate scale to the extreme right. If the column has pin ends this area (or weight) must be multiplied by the percentage factor found on the scale above the column ratio. The other scales are used in accordance with the foregoing descriptions.

It is generally not advisable to employ columns with a higher ratio of slenderness than 120. The "Code" (N. Y. C.) limits important columns to this ratio as a maximum, and the allowed stresses for columns are given only for ratios of 120 and less. As it was considered advisable to extend the use of Diagram No. 33 beyond this point so as to include ratios up to 200 the following method was used to supply safe values for allowable unit stress in case of ratios between 120 and 200. Different well established column formulas were plotted and the curve representing the unit stresses allowed by the "Code" was extended parallel to the direction of the mean of the other curves. The resulting curve was used to get safe allowable unit stresses for ratios beyond 120.

SECTION AREAS FOR ECCENTRIC LOADING:—It will be remembered that Chapter II contains discussions on the strength of columns under concentric and eccentric loading. The term $z\ y$—there defined as the coefficient of eccentricity—when divided by the square of the radius of gyration about the axis for which the load is eccentric gives a percentage factor for the area necessary to take care of the bending moment due to this eccentricity.

NOTE:—A little care in the arrangement of beams and girders will often eliminate the eccentricity of loading on columns. There are, however, many cases where it cannot be avoided. For such cases the "Code" (N. Y. C.) provides that:

Any column eccentrically loaded shall have the stresses caused by such eccentricity computed, and the combined stresses resulting from such eccentricity at any part of the column, added to all other stresses at that part shall in no case exceed the working stresses stated in the "Code." The eccentric load of a column shall be considered to be distributed equally over the entire area of that column at the next point below at which the column is securely braced laterally in the direction of the eccentricity.

It will be apparent that these provisions have been adhered to in the treatment of eccentric loads herewith presented.

DIAGRAM NO. 34 gives the aforementioned percentage of area necessary to take care of eccentricity of loading. In this diagram abscissas represent the radius of gyration, while ordinates represent coefficient of eccentricity. Curves on the diagram represent the percentage of area to be added to a cross section for the eccentricity of the loading on the column. An example will best illustrate the use of Diagrams Nos. 33 and 34.

Example:—A column 10 ft. long has a load of 140 tons, 15 tons of which is located 5 ins. from each neutral axis. The column section is built up with plates and angles in the form of an H. The assumed dimension back to back of angles is 10 ins., and 8 ins. is the dimension the other way.

Solution:—The center of gravity of the combined concentric and eccentric load is located 0.535 in. from both principal axis. The coefficient of eccentricity in the direction parallel to the web is 0.535 times 5 or 2.68; in the direction perpendicular to the web it is 0.535 times 4 or 2.14. For the direction parallel to the web in which case the radius is 4 ins., the diagram No. 33 gives the area of metal required for this eccentricity as 17% of what would be required for the same load concentrically located. For the direction perpendicular to the web in which case the radius is 2 ins., it is 53% of that same area. The sum of these three areas which go to make up the material of the cross section can be found in two ways from these diagrams.

First:—The area required for a concentric load of 140 tons may be found on Diagram No. 33 and the foregoing 70% (17 + 53) can then be added to it, giving the total area of cross section.

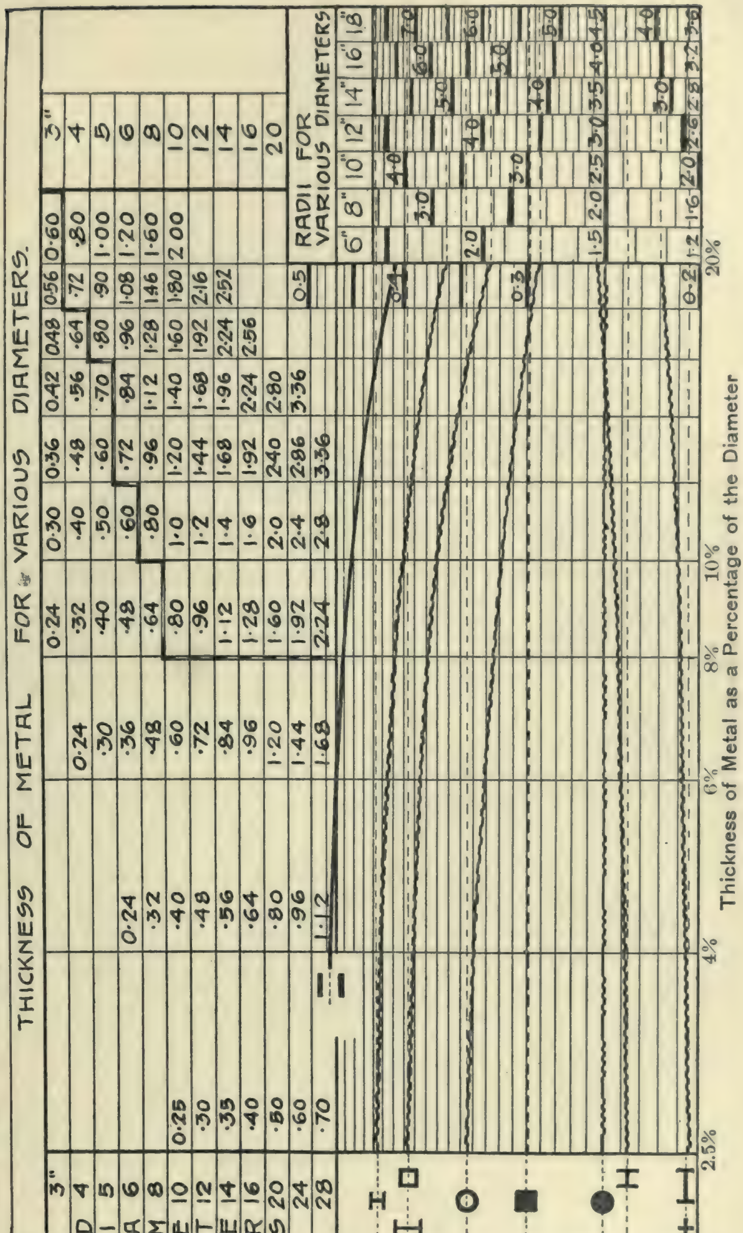
Second:—The load may be increased to what would be an equivalent concentric load—170% of 140 or 238 tons. Evidently the area may then be found directly on Diagram No. 33. This same diagram also gives the equivalent weight per lineal foot of the section. For a concentric load of 140 tons and a ratio of slenderness of 60 this weight is 81.5 lbs. and for a load of 238 tons, the weight of the cross section is about 139 lbs.

According to this diagram 139 lbs. per lin. ft. represents 41 sq. ins. of cross section. This would require approximately 1¾ in. thickness of metal for the assumed dimensions of the section, thus, it would be advisable to increase the dimensions until the thickness is much reduced.

The areas and weights, of structural shapes used for column sections, are given in the next chapter.

Diagram No. 30

For Giving the Radius of Gyration of the Most Common Forms of Column Sections of Wood, Cast Iron or Steel



Example.—The radius of gyration of a round column about ¾-inch metal is 3.25 for 10 ins. diameter.

Diagram No. 31

For giving the radius of gyration of the most common forms of built-up column sections.

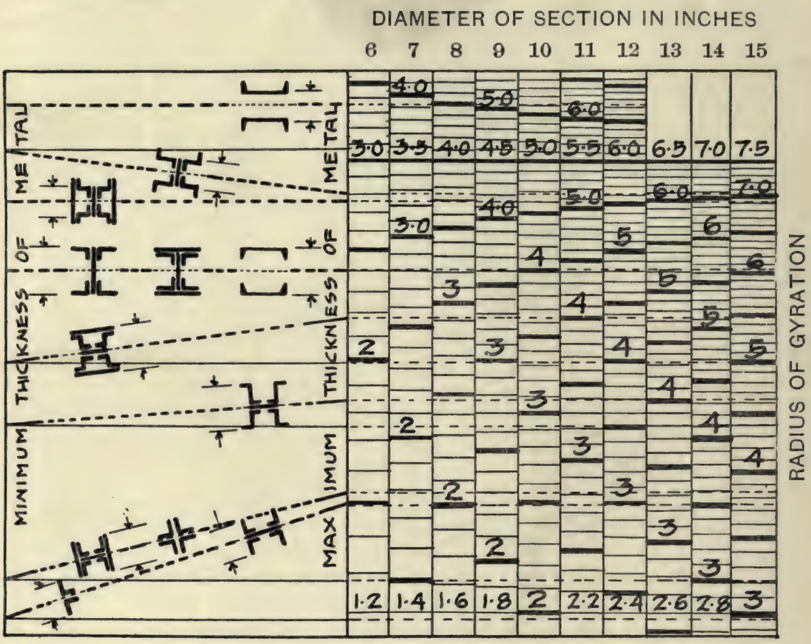
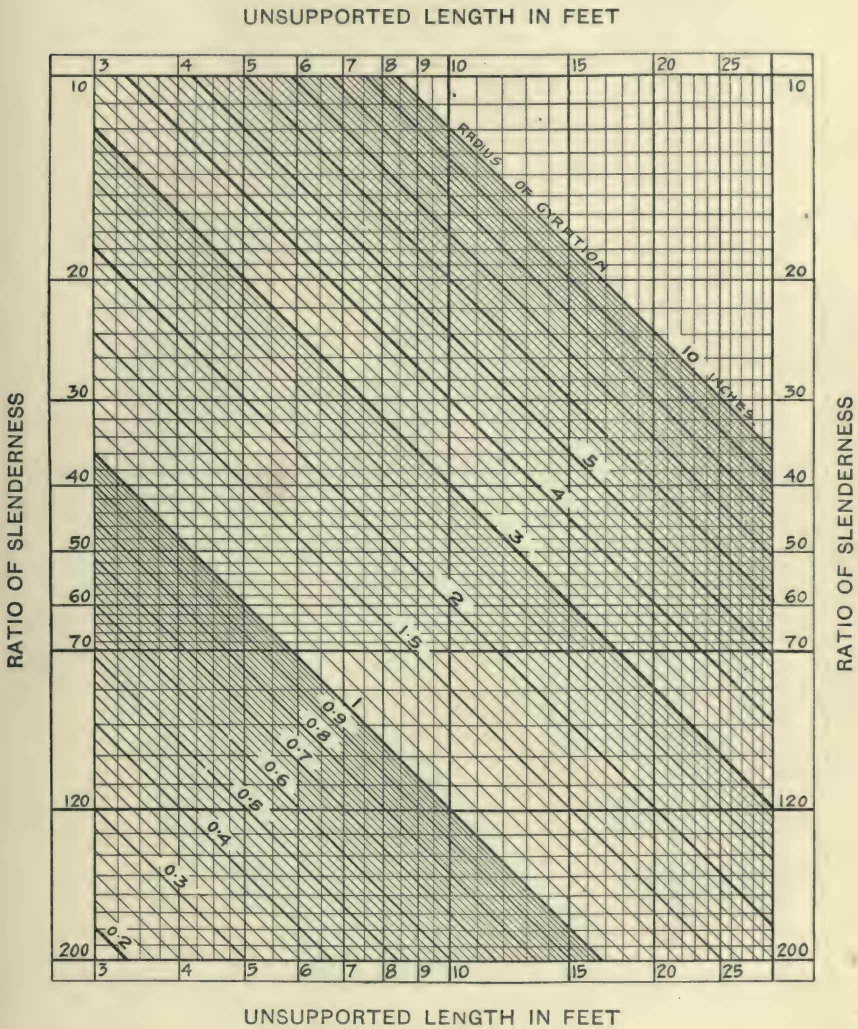


Diagram No. 32

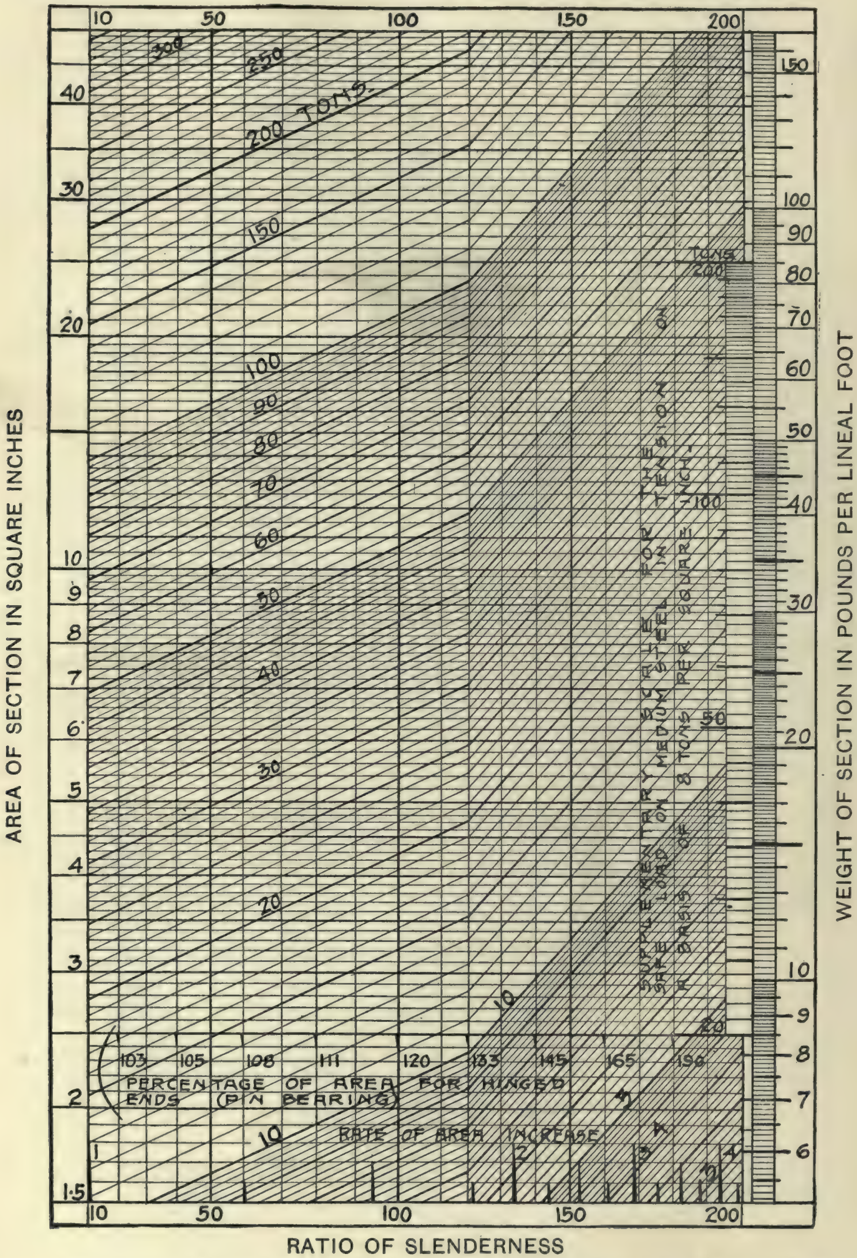
For giving the ratio of slenderness of a column.



Example.—A column 12 ft. long with a radius of gyration of 2.6 has a ratio of slenderness of 55.

Diagram No. 33

For giving the safe loads on steel columns as called for by the New York Building Code for ratios of slenderness up to 120, and as recommended by the author for ratios between 120 and 200.

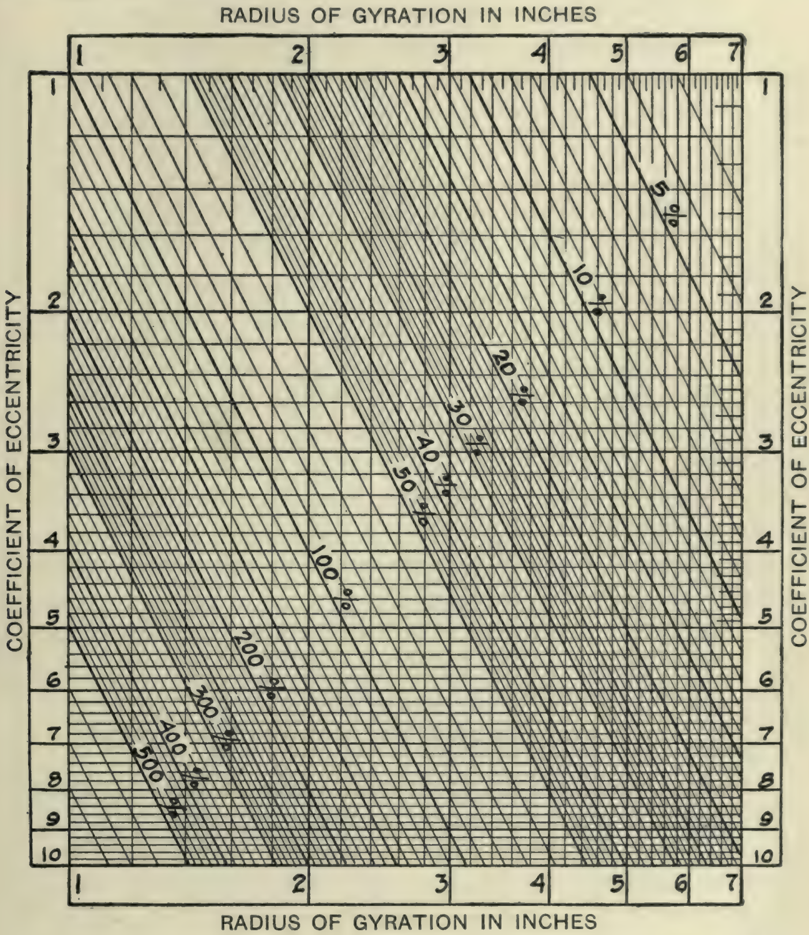


Example.—A column having a ratio of slenderness of 70 and a load of 54 tons requires 9.8 sq. ins. in its cross-section (or 33.3 lbs. per lin. foot).

Diagram No. 34

For Eccentric Loading on Columns.

This diagram gives the percentage of material necessary to add to the cross-section of a column for any eccentricity of the centre of gravity of its load with reference to the axial planes through its neutral axis.



Note.—The material in a column-section with two principal axes of symmetry—as in Figs. 3 to 23—may be said to perform three functions when the load on the column is eccentric: Part cares for the load as purely concentric, another part cares for the load as eccentric on the axis where the radius of gyration is a minimum, and the remainder on the axis where the radius is a maximum.

CHAPTER VIII. TABLES.

PROPERTIES OF SINGLE AND BUILT-UP STEEL SHAPES OF I-BEAMS, CHANNELS, ANGLES, TEES, ZEES, AND FLATS FOR USE IN COLUMNS, BEAMS AND TRUSSES.

EXPLANATIONS:—The tables of properties in this chapter while intended to cover the whole range of needs in structural design, are grouped with the treatment of column design for two reasons; first, the subject of beam design is so thoroughly covered by independent diagram treatment, that only occasional use will be made of beam properties; second, the subject of column design, being impossible of independent diagram treatment because of the complex forms of built sections, makes necessary a combination of diagrams and tables. In the first case the use of independent diagrams was made possible by the simplicity of form of the standard section, the I-beam (occasional use of channels, angles and tees excepted), being used almost exclusively. On the other hand, columns have only to a very small extent become standardized as regards section; practically all steel columns are built sections, i e. are built up of elementary structural shapes, and their proportions and sizes vary widely. This is one of the conditions that complicates column design and makes it impossible of independent diagram treatment.

The properties of the standard sections given in the tables are for the single shapes, for a pair of shapes, and also, in the case of angles and zees, for four shapes combined. In the latter case as well as in the cases of pairs of I-beams or pairs of channels these combined shapes are supposed to be latticed* together.

The aforementioned standard sections,† adopted by the American Association of Steel Manufacturers, as represented by Carnegie, Cambria, Jones and Laughlins and Phoenix are given in these tables; also distinctive and well marked differences in the Passaic and Pencoyd standards have been taken into account.

*When plates are used instead of lattice bars, the radius of gyration should be taken from the diagrams in the preceding chapter and not from these tables, except when special tables are appended with these values which is the case for channel and zee-bar columns. These give correct values for radius of gyration about both axes for cover and web plates respectively.

†See note in Chapter III.

NOTE:—The first mentioned group of rolling mills do not in every case roll the full list given in the tables, but differences of this kind have not been noted.

DIMENSIONS AND WEIGHTS:—The dimensions and weights per lineal foot given for all the sections in the tables are fully explained under their respective heads. However, it should be mentioned that the "Practical Detailing Dimensions" given in most of these Tables are not to be considered as arbitrary values—they represent mean values. The gage given for the I-beam is the distance between the two lines of rivets on the flanges. Thus, the distance from the center line of the web to the rivet line in the flange on either side is equal to one-half of the gage given in the table. The gage for channels is given from their back. Gages for a single and double line of rivets on the legs of angles are given in the table of angles with even legs. The first dimension in each case is given for the gage of a single line of rivets and directly below it is given the gages for a double line of rivets for the same leg.

As stated in the foregoing the range of application of these tables includes: beams and columns used in buildings; tension and compression members used in trusses; and flanges used in plate girders. These several uses will now be considered under their separate heads.

BEAMS.

The more generally used beam property is the section-moment. It is given in all of the following tables for the two principle axes of each section in foot-pounds for angles and tees and foot-tons for all other shapes. This is the safe resisting property of a beam which opposes the bending moment of the external forces acting upon it; and the Formulas Nos. 1 to 14, given in Chapter I are for the purpose of illustrating the application of this section-moment in the design of beams or girders.

The allowable values for the web of I-beams in compression and for the end reactions on beams have been fully described in Chapters I and V and the use of these values will be readily understood without further explanations.

When it is Desired to Use Plates on the Top and Bottom Flanges of an I-Beam or a pair of I-beams, or on the top and bottom flanges of a pair of channels, the following method may be employed to determine the area of the plates:

From the external bending moment deduct 75% of the section-moment

of the beam or beams (given in the tables) and divide this by the product of the depth (c. to c. of plates in feet) and 7.2. (Formula—22)

This quotient gives the net area of the plates on each flange. Diagram No. 33 may be used to find an equivalent weight per lineal foot for such an area of metal and Table No. 25 will give the several possible dimensions for the width and thickness of these plates.

TRUSSES AND PLATE GIRDERS.

The theory and practice of the design of trusses and plate girders (including the subject of rivet spacing) is beyond the scope of the present book and in the following a general knowledge of the subject, on the part of the reader, is assumed.

In the Design of Trusses, pure compression and pure tension usually prevail in both the web and chord members. However, when loads occur on the top and bottom chords, flexure stresses are thereby added to these direct stresses. For the design of members in pure tension and pure compression, no further explanations will be required than have already been given; while for cases of combined stresses of flexure and tension or flexure and compression the diagram for eccentric loading may be utilized in a very simple manner:—

For instance, the eccentric load P (see Fig. 1) considered in the subject of column design now becomes a transverse instead of an axial load on the truss member, and this transverse load produces a similar bending moment on the member. This bending moment due to transverse loading is found by introducing a new value for z, which is obtained as follows: Suppose, a and b are the respective distances of the load from the two ends of the member, then, z equals $\frac{a\,b}{a+b}$ for any position of the load on the truss member.

Therefore, the percentage of area of the cross section necessary to take care of the flexure stress (either tension or compression) can be found by the use of Diagram No. 34, after having determined upon the value of the radius of gyration of the proposed section, either from the following tables or from Diagram Nos. 30 or 31. The coefficient of eccentricity being $\frac{a\,b}{a+b}\,y$ instead of z y **as in**

the case of columns where y is the distance from the neutral axis to the extreme fiber, as before. Thus $A = \frac{P}{S}\left\{\frac{\frac{a\,b}{a+b}\,y}{r^2}\right\}$ (23).

When the load P is in the middle of the beam and both ends are fixed, the area will be one-half that given by the above formula.

The use of the diagrams and tables for tension and compression members has already been described in the preceding chapter on steel column design.

NOTE:—Deductions for rivet holes in tension members may be made as follows: The area of metal required for a ¾-in. rivet is 0.88 sq. ins. for every inch thickness of metal—this is equivalent to a reduction of 3 lbs. per lin. ft. in the weight of the section. Thus, if the metal is ½ in. thick, 1½ lbs. per lin. ft. should be added to the net section. Accurate values for various thicknesses of metal from ¼ in. to 2 ins. are given in Table No. 25 for ½ in. and ¾ in. rivets.

PLATE GIRDERS:—A general discussion of the subject of plate girder design is beyond the scope of this book. The tables given herewith, however, lend themselves readily to the design of an important element of plate girders—the flanges.

The area of the flange is found by the use of the following formula when the value of the section-moment in the web is neglected:—

$$F_a = 0.143\,\frac{M_b}{h} \qquad \textbf{(24a)}$$

where

F_a = the net area of the tension flange in square inches,
M_b = the external bending moment in foot-tons.
and h = the distance, in feet, between centers of gravity of flanges.

Now, as the weight per lineal foot of pairs of angles is given in the tables in preference to the area of the sections, this formula becomes

$$F_w = 0.485\,\frac{M_b}{h}, \qquad (24)$$

where

F_w = the weight in pounds per lineal foot of the tension flange, and the other values same as foregoing.

It is customary to make the compression flange of a plate girder the same as the tension flange.

COLUMNS MADE OF BUILT STEEL SHAPES.

In Figs. 3 to 18 herewith, are exhibited a number of built steel column sections. They do not represent every possible variation in arrangement of material; as a matter of fact it is not desirable to more than indicate the general methods of distributing the material in a cross section, because the system presented in this book


Fig. 3.


Fig. 4.


Fig. 5.

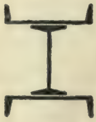


Fig. 6.

for the design of columns admits of any arrangement or distribution of the material that the ingenuity of the designer may dictate. This fact will be better understood from a description of the various types of column sections*:—

Combinations with I-Beams:—Figs. 3 to 6 refer to the use of an I-beam to combine channels, I-beams or plates, etc. For a single I-beam, and a pair of latticed I-beams, the tables given in this chapter are complete. When an I-beam is used to connect a pair of I-beams or a pair of channels, as in Figs. 5 and 6, special tables may be easily made by the designer. The only values required in these special tables being the sum of the weights of the combined sections, and the dimensions with which to determine the radii of gyration from Diagrams Nos. 30 and 31.

Channel Columns:—This type of column, Figs. 7, 8, and 9,

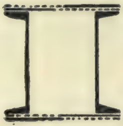


Fig. 7.


Fig. 8.


Fig. 9.

is the most popular of the closed sections. For this reason a special table has been appended to the general table of properties of channels. The properties of a pair of latticed channels are given in the general table. When plates are added to the webs of the channels as in Fig. 9 special tables may be made.

*No attempt is made to enter into a discussion of the relative merits of column sections. This aspect of the subject is taken up in books like Frietag's "Architectural Engineering" and books by Birkmire.

Plate and Angle Columns:—Figs. 10 to 16 illustrate in a small way the various combinations that may be made by the use of angles and plates. The type of column indicated by Figs. 10 and 11 is most popular with advocates of open sections. For this reason a special table is also appended to the tables of properties

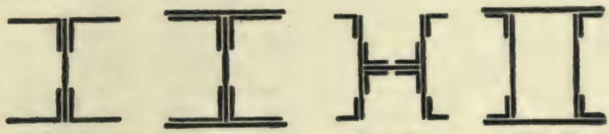


Fig. 10. Fig. 11. Fig. 12. Fig. 13.

Fig. 14. Fig. 15. Fig. 16.

of angles. Evidently other special tables can be made to cover an endless variety of combinations of these two sections.

Zee-Bar Columns:—Figs. 17 and 18 illustrate two of the more common forms of this type of column. Table No. 24 though small gives quite a variety of information on these column sections. For instance: Two methods of design are considered, one giving weight of section for varying widths of web and the other for a constant width of web. This latter is a favorite idea with some designers, giving as it does, one important constant dimension for

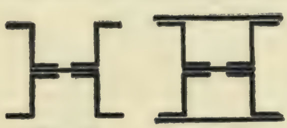


Fig. 17. Fig. 18.

the column. Other special tables can also be made for any other desired arrangement of material.

Miscellaneous Sections:—The several tables on I-beams, channels, tees, and angles may evidently be used for the application of any single shape to strut design.

STEEL I-BEAMS

For Beams, Girders, Columns, or Truss Members TABLE 17

		1	2	3	4	5	6	7	8
			Weight Per Foot		Dimensions			Neutral Axis Perpendicular to Web	
Key Number	Manufacturer	Height of Beam	One Beam	Two Beams	Flange Width	Web Thickness	Area of Section of One Beam	Radius of Gyration	Section-Moment
		Inches	Lbs.	Lbs.	Inches	Inches	Square Inches	Inches	Foot-Tons
1	ac	3	5.5	11	2.33	0.17	1.62	1.2	1.1
2	ac		6.5	13	2.42	0.26	1.91	1.2	1.2
3	ac		7.5	15	2.52	0.36	2.20	1.1	1.3
4	b	4	6.0	12	2.19	0.18	1.76	1.6	1.5
5	d		7.5	15	2.66	0.19	2.20	1.6	2.0
6	ac		8.5	17	2.73	0.26	2.50	1.6	2.1
7	ac		9.5	19	2.81	0.34	2.79	1.5	2.2
8	b		10	20	2.69	0.39	2.94	1.5	2.3
9	ac		10.5	21	2.88	0.41	3.09	1.5	2.4
10	d	5	9.75	19.5	3.00	0.21	2.87	2.1	3.2
11	b		12	24	3.13	0.34	3.53	2.0	3.6
12	ac		12.25	24.5	3.15	0.36	3.60	1.9	3.6
13	b		13	26	3.13	0.26	3.82	2.1	4.2
14	ac		14.75	29.5	3.29	0.50	4.34	1.9	4.0
15	b		15	30	3.25	0.38	4.41	2.0	4.5
16	b	6	12	24	3.38	0.22	3.53	2.5	4.8
17	ac		12.25	24.5	3.33	0.23	3.60	2.5	4.8
18	ac		14.75	29.5	3.45	0.35	4.34	2.4	5.3
19	b		15	30	3.52	0.25	4.41	2.5	5.9
20	ac		17.25	34.5	3.57	0.48	5.07	2.3	5.8
21	b		17.5	35	3.64	0.37	5.14	2.4	6.4
22	b		20	40	3.77	0.50	5.88	2.3	6.9
23	c		32.3		4.88	0.50	9.49	2.3	11.5
24	c		37.4				10.99	2.3	12.5
25	c		41		5.25	0.63	12.65	2.3	14.2
26	c		46.1				13.55	2.3	15.2
27	d	7	15	30	3.66	0.25	4.41	2.9	6.9
28	d		17.5	35	3.76	0.35	5.14	2.8	7.5
29	d		20	40	3.87	0.46	5.88	2.7	8.0
30	b		22	44	4.17	0.36	6.47	2.8	9.5
31	d	8	18	36	4.00	0.27	5.29	3.3	9.5
32	b		20	40	4.20	0.32	5.88	3.2	10.0
33	ac		20.5	41	4.08	0.36	6.02	3.2	10.1
34	d		22	44	4.38	0.29	6.47	3.3	11.6
35	ac		23	46	4.18	0.45	6.76	3.1	10.7
36	b		25	50	4.49	0.40	7.35	3.2	12.4
37	ac		25.5	51	4.27	0.54	7.49	3.0	11.4
38	b		27	54	4.56	0.48	7.93	3.1	12.9
39	d	9	21	42	4.33	0.29	6.17	3.7	12.6
40	b		23.3	46.6	4.58	0.35	6.85	3.6	13.2
41	d		25	50	4.45	0.41	7.35	3.5	13.6
42	b		27	54	4.75	0.31	7.93	3.7	16.4
43	d		30	60	4.61	0.57	8.82	3.4	15.1
44	b		33	66	4.95	0.51	9.70	3.5	18.1
45	ac		35	70	4.77	0.73	10.29	3.3	16.5

a denotes beams rolled by Carnegie, Cambria, Jones & L., and Phœnix; *b* by Passaic; *c* by Pencoyd; *d* by all mills (includes *a, b, c*).

Note:—No. 31 by Jones & L., and Nos. 33, 35 and 37 by Jones & L. and Cambria are rolled with ¼ lb. less metal per foot than weights given in above table.

STEEL I-BEAMS—Continued

For Beams, Girders, Columns, or Truss Members TABLE 17

Key Number	9	10	11	12	13	14	15	16
	Neutral Axis Coincident with Web		Distance C. to C. of Webs required to make Radii of Gyration Equal	Allowable End Reaction	Web in Compression. Allowable load per lin. ft. under Column Base or other Load.	Practical Detailing Dimensions		
							Flange	
	Radius of Gyration	Section-Moment				Clear distance between fillets on web	Maxim. Diam. of Rivet	Gage of Rivet Lines
	Inches	Foot-Tons	Inches	Tons	Tons	Inches	Inches	Inches
1	**0.5**	**0.26**		**2.3**	**13.2**		⅜	1¼
2	0.5	0.29		3.5	21.5		to	to
3	0.5	0.32		4.8	31.5	1⅞	½	1½
4	0.5	0.23		3.2	13.1			
5	**0.6**	**0.38**		**3.4**	**14.0**			1½
6	0.6	0.41		4.7	20.4		½	to
7	0.6	0.44		6.1	27.6			1¾
8	0.6	0.44		7.0	32.0			
9	0.6	0.46		7.4	34.0	2¼		
10	**0.7**	**0.54**		**4.7**	**14.8**			
11	0.6	0.61		7.6	26.6			
12	0.6	0.61		8.1	28.5		½	1¾
13	0.7	0.84		5.8	19.3		to	to
14	0.6	0.69		11.2	41.3		⅝	2
15	0.7	0.92		8.5	30.2	3⅝		
16	0.7	0.75	4.7	5.9	14.6			
17	**0.7**	**0.74**		**6.2**	**15.5**			
18	0.7	0.81		9.4	26.6			
19	0.8	1.04		6.7	17.5		⅝	1¾
20	0.7	0.88	4.3	12.9	38.3	4½		to
21	0.8	1.11		10.0	28.3			2
22	0.8	1.20		13.5	40.0			
23	1.1	3.2				3	⅝ to ¾	3
24	1.1							
25	1.2	4.6				2⅝	¾ to ⅞	3¼
26	1.2							
27	**0.8**	**1.0**	**5.5**	**7.9**	**16.4**			
28	0.8	1.0		11.0	25.5		⅝	2
29	0.7	1.1	5.2	14.5	35.5	5⅜	to	to
30	0.9	1.7		11,3	26.5		¾	2¼
31	**0.8**	**1.3**	**6.3**	**9.2**	**17.1**			
32	0.9	1.4		11.5	21.7			
33	0.8	1.3		13.0	25.4			2¼
34	1.0	1.8		10.3	19.0		¾	to
35	0.8	1.4	6.0	16.2	33.5			2½
36	0.9	1.9		14.4	28.8			
37	0.8	1.5	5.8	19.5	41.8	6¼		
38	0.9	2.0		17.3	36.2			
39	**0.9**	**1.6**	**7.1**	**10.5**	**17.8**			
40	0.9	1.7		14.2	23.4			
41	0.9	1.7		16.6	28.8			2½
42	1.1	2.5		11.9	19.8		¾	to
43	0.8	1.8		23.1	43.5			2¾
44	1.0	2.8		20.7	38.0			
45	0.8	2.0	6.4	29.6	58.0	$7\frac{1}{16}$		

STEEL I-BEAMS

For Beams, Girders, Columns, or Truss Members TABLE 17

Key Number	Manufacturer	1	2	3	4	5	6	7	8
		Height of Beam	Weight Per Foot		Dimensions		Area of Section of One Beam	Neutral Axis Perpendicular to Web	
			One Beam	Two Beams	Flange Width	Web Thickness		Radius of Gyration	Section-Moment
		Inches	Lbs.	Lbs.	Inches	Inches	Square Inches	Inches	Foot-Tons
46	d	**10**	**25**	**50**	**4.66**	**0.31**	**7.35**	**4.1**	**16.3**
47	b		27	54	4.81	0.37	7.93	4.0	17.0
48	d		30	60	4.81	0.46	8.82	3.9	17.9
49	b		33	66	5.00	0.37	9.70	4.1	21.5
50	d		35	70	4.95	0.60	10.29	3.8	19.5
51	d		40	80	5.10	0.75	11.75	3.7	21.2
52	d	**12**	**31.5**	**63**	**5.00**	**0.35**	**9.26**	**4.8**	**24.0**
53	d		35	70	5.09	0.44	10.29	4.7	25.4
54	d		**40**	**80**	**5.25**	**0.46**	**11.75**	**4.8**	**29.9**
55	d		45	90	5.37	0.58	13.22	4.6	31.7
56	d		50	100	5.49	0.70	14.69	4.5	33.7
57	d		55	110	5.61	0.82	16.16	4.4	35.7
58	bc		60	120	6.12	0.75	17.63	4.6	41.7
59	bc		65	130	6.25	0.88	19.10	4.5	43.7
60	d	**15**	**42**	**84**	**5.50**	**0.41**	**12.34**	**6.0**	**39.3**
61	d		45	90	5.55	0.46	13.22	5.9	40.5
62	d		50	100	5.65	0.56	14.69	5.7	43.0
63	d		55	110	5.75	0.66	16.16	5.6	45.4
64	d		**60**	**120**	**6.00**	**0.59**	**17.63**	**5.9**	**54.1**
65	d		65	130	6.10	0.69	19.10	5.8	56.5
66	d		70	140	6.19	0.78	20.57	5.7	59.0
67	d		75	150	6.29	0.88	22.04	5.6	61.4
68	d		**80**	**160**	**6.40**	**0.81**	**23.51**	**5.8**	**70.7**
69	Carnegie		85	170	6.48	0.89	24.98	5.7	72.7
70	"		90	180	6.58	0.99	26.45	5.6	75.1
71	"		95	190	6.68	1.09	27.92	5.6	77.6
72	"		100	200	6.78	1.19	29.39	5.5	80.0
73	d	**18**	**55**	**110**	**6.00**	**0.46**	**16.16**	**7.1**	**58.9**
74	d		60	120	6.10	0.56	17.63	6.9	62.4
75	d		65	130	6.18	0.64	19.10	6.8	65.3
76	d		70	140	6.26	0.72	20.57	6.7	68.2
77	bc		75	150	6.16	0.66	22.04	7.0	80.8
78	bc		80	160	6.38	0.69	23.51	6.9	83.8
79	c		85	170	7.00	0.74	24.98	6.8	85.2
80	c		90	180	7.08	0.82	26.45	6.7	88.0
81	d	**20**	**65**	**130**	**6.25**	**0.50**	**19.10**	**7.8**	**78.0**
82	d		70	140	6.33	0.58	20.57	7.7	81.3
83	d		75	150	6.40	0.65	22.04	7.6	84.6
84	d		**80**	**160**	**7.00**	**0.60**	**23.51**	**7.9**	**97.8**
85	d		85	170	7.06	0.66	24.98	7.8	100.6
86	d		90	180	7.14	0.74	26.45	7.7	103.9
87	ac		95	190	7.21	0.81	27.92	7.6	107.1
88	ac		100	200	7.28	0.88	29.39	7.5	110.4
89	ac	**24**	**80**	**160**	**7.00**	**0.50**	**23.51**	**9.5**	**116.0**
90	ac		85	170	7.07	0.57	24.98	9.3	120.5
91	ac		90	180	7.13	0.63	26.45	9.2	124.4
92	ac		95	190	7.19	0.69	27.92	9.1	128.3
93	ac		100	200	7.25	0.75	29.39	9.0	132.2

a denotes beams rolled by Carnegie, Cambria, Jones & L., and Phœnix; *b* by Passaic; *c* by Pencoyd; *d* by all mills (includes *a, b, c*).

STEEL I-BEAMS—Continued

For Beams, Girders, Columns, or Truss Members TABLE 17

Key Number	9	10	11	12	13	14	15	16
	Neutral Axis Coincident with Web		Distance C. to C. of Webs required to make Radii of Gyration Equal	Allowable End Reaction	Web in Compression per Lin. Foot	Practical Detailing Dimensions		
							Flange	
	Radius of Gyration	Section-Moment				Clear distance between fillets on web	Maxim. Diam. of Rivet	Gage of Rivet Lines
	Inches	Foot-Tons	Inches	Tons	Tons	Inches	Inches	Inches
46	**1.0**	**2.0**	**7.9**	**12.0**	**18.6**			
47	1.0	2.1		16.4	24.6			
48	0.9	2.1		20.7	32.4			2½
49	1.1	3.1		16.4	24.6		¾	to
50	0.9	2.3		27.0	45.2			3
51	0.9	2.5	7.1	33.8	58.8	7½		
52	**1.0**	**2.5**	**9.5**	**15.6**	**20.0**			
53	1.0	2.8		23.8	28.5			
54	**1.1**	**3.5**	**9.3**	**24.9**	**30.0**			
55	1.1	3.7		31.3	41.0		¾	2¾
56	1.1	3.9		37.8	52.0		to	to
57	1.0	4.1	8.7	44.3	63.0	8⅝	⅞	3½
58	1.2	5.6		40.5	56.5			
59	1.2	6.1		47.5	68.5			
60	**1.1**	**3.5**	**11.7**	**19.1**	**22.3**			
61	1.1	3.6		25.5	26.6			3
62	1.0	3.8		37.2	36.0			to
63	1.0	4.0	11.1	44.6	45.0			3½
64	**1.2**	**5.8**	**11.5**	**39.9**	**38.8**	11¾		
65	1.2	6.0		46.6	47.9		¾	3¼
66	1.2	6.2		52.7	56.2		to	to
67	1.2	6.5	11.0	59.4	65.5		⅞	4
68	**1.3**	**8.7**	**11.3**	**54.7**	**59 0**			
69	1.3	9.0		60.1	66.0			3¾
70	1.3	9.3		66.9	75.0			to
71	1.3	9.6		73.6	84.5			4
72	1.3	10.0	10.8	79.7	93.2	10¾		
73	**1.2**	**4.7**	**14.0**	**22.4**	**23.4**			
74	1.1	4.9		36.2	33.0			
75	1.1	5.1		49.4	40.0			
76	1.1	5.2	13.2	58.3	47.6	14¾	¾	3¼
77	1.3	8.1		46.2	46.0		to	to
78	1.3	8.2		56.7	53.2		⅞	4
79	1.3	8.4		59.9	48.5			
80	1.3	8.7		66.4	56.0			
81	**1.2**	**5.9**	**15.5**	**25.0**	**25.0**			
82	1.2	6.1		39.9	32.5			
83	1.2	6.3	15.0	50.5	38.7			
84	**1.4**	**8.7**	**15.5**	**41 8**	**34.2**			3½
85	1.4	8.9		51.7	39.5		⅞	to
86	1.4	9.1		64.0	47.3			4¼
87	1.3	9.4		72.9	53.5			
88	1.3	9.6	14.8	79.2	60.0	16		
89	**1.4**	**8.2**	**18.7**	**20.4**	**19.2**			
90	1.3	8.3		30.9	26.0		⅞	3¾
91	1.3	8.5		42.3	32.0		to	to
92	1.3	8.7		54.5	37.5		1	4½
93	1.3	8.9	17.8	66.3	43.2	20½		

STEEL CHANNELS

For Beams, Girders, Columns, or Truss Members TABLE 18

		1	2	3	4	5	6	7	8
			Weight per foot		Dimensions			Neutral axis Perpendicular to Web	
Key number	Manufacturer	Height of Channel	One Channel	Two Channels	Flange Width	Web Thickness	Area of Section of one Channel	Radius of Gyration	Section-Moment 8 tons per □" Max. fibre stress
		Inches	Lbs.	Lbs.	Inches	Inches	Square Inches	Inches	Foot-Tons
1	ac	3	4	8	1.41	0.17	1.17	1.2	0.7
2	ac		5	10	1.50	26	1.47	1.1	0.8
3	ac		6	12	1.60	36	1.76	1.1	0.9
4	b	4	5	10	1.59	0.17	1.47	1.6	1.2
5	ac		5.25	10.5	1.58	0.18	1.54	1.6	1.3
6	b		6	12	1.66	0.24	1.76	1.5	1.3
7	ac		6.25	12.5	1.65	0.25	1.84	1.5	1.4
8	ac		7.25	14.5	1.73	0.33	2.13	1.5	1.5
9	b		8	16	1.86	0.27	2.35	1.5	1.8
10	b		10	20	2.01	0.42	2.94	1.5	2.1
11	b	5	6	12	1.66	0.18	1.76	1.9	1.7
12	ac		6.5	13	1.75	0.19	1.91	1.9	2.0
13	b		8	16	1.78	0.30	2.35	1.8	2.1
14	d		9	18	1.89	0.33	2.64	1.8	2.3
15	b		10	20	1.97	0.31	2.94	1.9	2.7
16	ac		11.5	23	2.04	0.48	3.38	1.7	2.8
17	b		12	24	2.09	0.43	3.52	1.8	3.1
18	d	6	8	16	1.92	0.20	2.35	2.3	2.8
19	b		9	18	1.99	0.25	2.64	2.3	3.0
20	b		10	20	2.04	0.30	2.94	2.2	3.2
21	ac		10.5	21	2.04	0.32	3.08	2.2	3.3
22	b		12	24	2.19	0.28	3.52	2.3	4.1
23	d		13	26	2.16	0.44	3.82	2.1	3.9
24	b		15	30	2.34	0.43	4.41	2.2	4.7
25	ac		15.5	31	2.28	0.56	4.56	2.1	4.3
26	b		17	34	2.41	0.38	5.00	2.3	5.6
27	b		18	36	2.46	0.43	5.29	2.3	5.8
28	b		20	40	2.56	0.53	5.88	2.2	6.2
29	b	7	9	18	2.00	0.20	2.64	2.7	3.6
30	ac		9.75	19.5	2.09	0.21	2.86	2.7	4.0
31	b		10	20	2.04	0.24	2.94	2.6	3.8
22	b		12	24	2.13	0.33	3.52	2.6	4.3
33	ac		12.25	24.5	2.20	0.32	3.60	2.6	4.6
34	b		13	26	2.22	0.28	3.82	2.7	5.2
35	ac		14.75	29.5	2.30	0.42	4.34	2.5	5.2
36	b		15	30	2.30	0.36	4.41	2.6	5.6
37	b		17	34	2.39	0.45	5.00	2.5	6.1
38	ac		17.25	34.5	2.41	0.53	5.07	2.4	5.7
39	ac		19.75	39.5	2.51	0.63	5.80	2.4	6.3

a denotes beams rolled by Carnegie, Cambria, Jones & L., and Phœnix; *b* by Passaic; *c* by Pencoyd; *d* by all mills (includes *a, b, c*).

STEEL CHANNELS—Continued

For Beams, Girders, Columns, or Truss Members TABLE 18

Key Number	9	10	11	12	13	14	15	16	17
	Neutral Axis Parallel with Web					Practical Detailing Dimensions			
							Flange		Lattice Bars
	Radius of Gyration	Section-Moment 8 tons per □″ Max. fibre stress	Distance of Centre of Gravity from Back of Channel	Distance B. to B. required to make Radii of Gyration equal: Flanges Out	Flanges In	Clear Distance Between Fillets on Web	Maximum Diameter of Rivet	Gage of Rivet Lines	Size for Single Lacing, also Maximum and Minimum distance C. to C. of Rivets
	Inches	Foot-Tons	Inches	Inches	Inches	Inches	Inches	Inches	Ft. and Ins.
1	**0.41**	**0.14**	**0.44**	**1.3**					
2	0.41	0.16	0.44				⅜	⅞	
3	0.42	0.18	0.46	1.1		1¾			
4	0.45	0.17	0.46	2.0				⅞	
5	**0.45**	**0.19**	**0.46**						
6	0.46		0.45				⅜		
7	0.45	0.21	0.46				to		
8	0.46	0.23	0.46				½		
9	0.55	0.36	0.59						
10	0.55	0.42	0.60	1.8		2¾		1⅛	
11	0.47	0.21	0.45	2.8	4.6			⅞	
12	**0.50**	**0.25**	**0.49**						
13	0.46		0.44				½		
14	0.49	0.30	0.48						
15	0.56		0.57						
16	0.49	0.36	0.51						
17	0.56	0.49	0.58	2.3	4.6	3⅝		1 3/16	
18	**0.54**	**0.33**	**0.52**	**3.5**	**5.6**		½	1	1½″ x ¼″
19	0.55		0.51						C. to C:—
20	0.54		0.50						Mx.0′-11½″
21	0.53	0.38	0.50						Mn.0′-6⅝″
22	0.63	0.60	0.65						
23	0.53	0.43	0.52						
24	0.63		0.65						
25	0.53	0.49	0.55			4⅜			
26	0.70	0.98	0.78				½		
27	0.71		0.79				to		
28	0.72	1.11	0.80	2.6	5.8		⅝	1½	
29	0.56	0.37	0.51	4.3	6.3		⅝	1⅛	1¾″ x ¼″
30	**0.59**	**0.42**	**0.55**						C. to C:—
31	0.55		0.50						Mx. 1′-1½″
32	0.54		0.49						Mn. 0′-7⅝″
33	0.58	0.47	0.53						
34	0.63	0.63	0.62						
35	0.57	0.53	0.54						
36	0.63		0.62						
37	0.63	0.74	0.62				⅝		
38	0.56	0.58	0.56				to	1½	
39	0.56	0.64	0.58	3.5	6.0	5¼	¾		

STEEL CHANNELS

For Beams, Girders, Columns, or Truss Members — TABLE 18

		1	2	3	4	5	6	7	8
			Weight per foot		Dimensions			Neutral axis Perpendicular to Web	
Key number	Manufacturer.	Height of Channel	One Channel	Two Channels	Flange Width	Web Thickness	Area of Section of one Channel	Radius of Gyration	Section-Moment 8 tons per □" Max. fibre stress
		Inches	Lbs.	Lbs.	Inches	Inches	Square Inches	Inches	Foot-Tons
40	b	8	10	20	2.08	0.20	2.94	3.1	4.7
41	b		11	22	2.12	0.24	3.23	3.0	4.9
42	ac		**11.25**	**22.5**	**2.26**	**0.22**	**3.30**	**3.1**	**5.4**
43	b		12	24	2.15	0.27	3.52	3.0	5.3
44	b		13	26	2.22	0.25	3.82	3.1	5.9
45	ac		13.75	27.5	2.35	0.31	4.04	3.0	6.0
46	b		15	30	2.29	0.32	4.41	3.0	6.4
47	ac		16.25	32.5	2.44	0.40	4.77	2.9	6.7
48	b		17	34	2.37	0.40	5.00	2.9	7.0
49	ac		18.75	37.5	2.53	0.49	5.51	2.8	7.3
50	ac		21.25	42.5	2.62	0 58	6.24	2.8	7.9
51	b	9	13	26	2.36	0.23	3.82	3.5	6.7
52	ac		**13.25**	**26.5**	**2.43**	**0.23**	**3.89**	**3.5**	**7.0**
53	b		14	28	2.39	0.26	4.11	3.4	7.0
54	d		15	30	2.49	0.29	4.41	3.4	7.5
55	b		16	32	2.56	0.28	4.70	3.5	8.4
56	b		18	36	2.63	0.35	5.29	3.4	9.0
57	ac		20	40	2.65	0.45	5.88	3.2	9.0
58	b		21	42	2.73	0.45	6.17	3.3	9.9
59	ac		25	50	2.82	0.62	7.35	3.1	10.5
60	d	10	**15**	**30**	**2.60**	**0.24**	**4.41**	**3.9**	**8.9**
61	b		17	34	2.64	0.29	5.00	3.8	9.5
62	b		18	36	2.67	0.32	5.29	3.7	9.8
63	d		20	40	2.74	0.38	5.88	3.7	10.5
64	d		25	50	2.89	0.53	7.35	3.5	12.1
65	d		30	60	3.04	0.68	8.81	3.4	13.7
66	ac		35	70	3.18	0.82	10.29	3.4	15.4
67	b	12	20	40	2.88	0.28	5.88	4.6	13.8
68	ac		**20.5**	**41**	**2.94**	**0.28**	**6.02**	**4.6**	**14.3**
69	b		23	46	2.95	0.35	6.76	4.5	15.0
70	d		25	50	3.05	0.39	7.35	4.4	16.0
71	b		27	54	3.13	0.38	7.93	4.5	17.8
72	d		30	60	3.17	0.51	8.81	4.3	17.9
73	b		33	66	3.28	0.53	9.70	4.3	20.2
74	d		35	70	3.30	0.64	10.29	4.2	19.9
75	ac		40	80	3.42	0.76	11.75	4.1	21.9
76	d	15	**33**	**66**	**3.40**	**0.40**	**9.70**	**5.6**	**27.8**
77	d		35	70	3.43	0.43	10.29	5.6	28.5
78	d		40	80	3.52	0.52	11.75	5.4	30.9
79	d		45	90	3.62	0.62	13.22	5.3	33.3
80	d		50	100	3.72	0.72	14.69	5.2	35.8
81	ac		55	110	3.82	0.82	16.16	5.2	38.3

a denotes beams rolled by Carnegie, Cambria, Jones & L., and Phœnix; *b* by Passaic; *c* by Pencoyd; *d* by all mills (includes *a*, *b*, *c*).

STEEL CHANNELS—Continued

For Beams, Girders, Columns, or Truss Members — TABLE 18

	9	10	11	12	13	14	15	16	17
	Neutral Axis Parallel with Web					Practical Detailing Dimensions			
							Flange		Lattice Bars
Key Number	Radius of Gyration	Section-Moment 8 tons per □″ Max. fibre stress	Distance of Centre of Gravity from Back of Channel	Distance B. to B. required to make Radii of Gyration equal: Flanges Out	Distance B. to B. required to make Radii of Gyration equal: Flanges In	Clear Distance Between Fillets on Web	Maximum Diameter of Rivet	Gage of Rivet Lines	Size for Single Lacing, also Maximum and Minimum distance C. to C. of Rivets
	Inches	Foot-Tons	Inches	Inches	Inches	Inches	Inches	Inches	Feet and Ins.
40	0.58	0.43	0.52	5.0	7.1			1⅛	2″ x 5/16″
41	0.58		0.51						C. to C:—
42	**0.63**	**0.53**	**0.58**						Mx. 1′-3″
43	0.57		0.50						Mn. 0′-8¾″
44	0.62	0.60	0.58						
45	0.62	0.58	0.56				¾		
46	0.61		0.57						
47	0.61	0.63	0.56						
48	0.61	0.69	0.58					1⅜	
49	0.60	0.68	0.57						
50	0.60	0.74	0.59	4.2	6.7	6 1/16			
51	0.64	0.61	0.57	5.7	7.9			1¼	2″ x 5/16″
52	**0.67**	**0.65**	**0.16**						C. to C:—
53	0.65		0.57						Mx. 1′-4½″
54	0.67	0.69	0.59				¾		Mn. 0′-9½″
55	0.74	0.91	0.66						
56	0.72		0.66						
57	0.65	0.79	0.59						
58	0.71	1.03	0.66					1½	
59	0.64	0.91	0.62	4.8	7.6	6⅞			
60	**0.72**	**0.78**	**0.64**	**6.3**	**8.9**			1½	2″ x ⅜″
61	0.71		0.62						C. to C:—
62	0.71		0.61				¾		Mx. 1′-6½″
63	9.70	0.89	0.61						Mn. 0′-10¾″
64	0.68	1.00	0.62						
65	0.67	1.11	0.65					2	
66	0.67	1.25	0.70	5.2	8.3	7⅝			
67	0.79	1.12	0.69	7.7	10.4		¾	1⅝	2¼″ x ⅜″
68	**0.81**	**1.17**	**0.70**						C. to C:—
69	0.78		0.67						Mx. 1′-10½″
70	0.79	1.27	0.68						Mn. 1′-1″
71	0.86	1.61	0.78						
72	0.77	1.39	0.68						
73	0.84		0.77				¾		
74	0.76	1.51	0.69				to	2⅛	
75	0.75	1.64	0.72	6.6	9.9	9 3/16	⅞		
76	**0.91**	**2.11**	**0.79**	**9.5**	**12.7**			2⅛	2½″ x ⅜″
77	0.91	2.15	0.79						C. to C:—
78	0.89	2.29	0.78				¾		Mx. 2′-2½″
79	0.88	2.42	0.79				to		Mn. 1′-3¼″
80	0.87	2.57	0.80				⅞	2½	
81	0.87	2.72	0.82	8.5	11.7	11⅞			

PLATE AND CHANNEL COLUMNS

(Supplement to Table 18) TABLE 19

1	2	3	4	5	6	1	2	3	4	5	6
6″ CHANNELS						7″ CHANNELS					
		8″ Plates		9″ Plates				9″ Plates		11″ Plates	
Weight of each Channel	Thickness of Plates	Weight of Column	Least Radius of Gyration	Weight of Column	Radius of Gyration equal on both Axes	Weight of each Channel	Thickness of Plates	Weight of Column	Least Radius of Gyration	Weight of Column	Radius of Gyration equal on both Axes
Lbs. per ft.	Inches	Lbs. per ft.	Inches	Lbs. per ft.	Inches	Lbs. per ft.	Inches	Lbs. per ft	Inches	Lbs. per ft.	Inches
8	¼	29.6	2.3	31.3	2.7	**9.75**	¼	34.8	2.6	38.2	3.2
	5/16	33.0		35.1			5/16	38.6		42.9	
	⅜	36.4		39.0			⅜	42.5		47.6	
	7/16	39.8		42.8			7/16	46.3		52.2	
	½	43.2		46.6			½	50.1		56.9	
	9/16	46.6		50.4			9/16	53.9		61.5	
	⅝	50.0	2.3	54.3	2.7		⅝	57.8	2.6	66.3	3.3
10.5	¼	34.6	2.3	36.3	2.7	**12.25**	¼	39.8	2.5	43.2	3.1
	5/16	38.0		40.1			5/16	43.6		47.9	
	⅜	41.4		44.0			⅜	47.5		52.6	
	7/16	44.8		47.8			7/16	51.3		57.2	
	½	48.2		51.6			½	55.1		61.9	
	9/16	51.6		55.4			9/16	58.9		66.5	
	⅝	55.0	2.3	59.3	2.6		⅝	62.8	2.6	71.3	3.3
13	¼	39.6	2.2	41.3	2.5	**14.75**	¼	44.8	2.5	48.2	3.0
	5/16	43.0		45.1			5/16	48.6		52.9	
	⅜	46.4		49.0			⅜	52.5		57.6	
	7/16	49.8		52.8			7/16	56.3		62.2	
	½	53.2		56.6			½	60.1		66.9	
	9/16	56.6		60.4			9/16	63.9		71.5	
	⅝	60.0	2.2	64.4	2.6		⅝	67.8	2.5	76.3	3.3
15.5	¼	44.6	2.1	46.3	2.5	**17.25**	¼	49.8	2.4	53.2	2.9
	5/16	48.0		50.1			5/16	53.6		57.9	
	⅜	51.4		54.0			⅜	57.5		62.6	
	7/16	54.8		57.8			7/16	61.3		67.2	
	½	58.2		61.6			½	65.1		71.9	
	9/16	61.6		65.4			9/16	68.9		76.5	
	⅝	65.0	2.2	69.3	2.6		⅝	72.8	2.5	81.3	3.2
						19.75	¼	54.8	2.4	58.2	2.9
							5/16	58.6		62.9	
							⅜	62.5		67.6	
							7/16	66.3		72.2	
							½	70.1		76.9	
							9/16	73.9		81.5	
							⅝	77.8	2.4	86.3	3.2

PLATE AND CHANNEL COLUMNS (Continued)

(Supplement to Table 18) TABLE 19

1	2	3	4	5	6	1	2	3	4	5	6
8″ CHANNELS						9″ CHANNELS					
		10″ Plates		12″ Plates				11″ Plates		13″ Plates	
Weight of each Channel	Thickness of Plates	Weight of Column	Least Radius of Gyration	Weight of Column	Radius of Gyration equal on both Axes	Weight of each Channel	Thickness of Plates	Weight of Column	Least Radius of Gyration	Weight of Column	Radius of Gyration equal on both Axes
Lbs. per ft.	Inches	Lbs. per ft.	Inches	Lbs. per ft.	Inches	Lbs. per ft.	Inches	Lbs. per ft	Inches	Lbs. per ft.	Inches
11.25	¼	39.5	3.0	42.9	3.6	**13.25**	¼	45.2	3.3	48.6	4.1
	5/16	43.7		48.0			5/16	49.9		54.1	
	⅜	48.0		53.1			⅜	54.6		59.7	
	7/16	52.3		58.2			7/16	59.2		65.2	
	½	56.5		63.3			½	63.9		70.7	
	9/16	60.8		68.4			9/16	68.5		76.2	
	⅝	65.0	2.9	73.5	3.7		⅝	73.3	3.3	81.7	4.0
13.75	¼	44.5	2.9	47.9	3.5	**15**	¼	48.7	3.3	52.1	4.0
	5/16	48.7		53.0			5/16	53.4		57.6	
	⅜	53.0		58.1			⅜	58.1		63.2	
	7/16	57.3		63.2			7/16	62.7		68.7	
	½	61.5		68.3			½	67.4		74.2	
	9/16	65.8		73.4			9/16	72.0		79.7	
	⅝	70.0	2.9	78.5	3.6		⅝	76.8	3.3	85.2	4.0
16.25	¼	49.5	3.0	52.9	3.4	**20**	¼	58.7	3.2	62.1	3.8
	5/16	53.7		58.0			5/16	63.4		67.6	
	⅜	58.0		63.1			⅜	68.1		73.2	
	7/16	62.3		68.2			7/16	72.7		78.7	
	½	66.5		73.3			½	77.4		84.2	
	9/16	70.8		78.4			9/16	82.0		89.7	
	⅝	75.0	3.0	83.5	3.6		⅝	86.8	3.2	95.2	3.9
18.75	¼	54.5	2.8	57.9	3.3	**25**	¼	68.7	3.1	72.1	3.6
	5/16	58.7		63.0			5/16	73.4		77.6	
	⅜	63.0		68.1			⅜	78.1		83.2	
	7/16	67.3		73.2			7/16	82.7		88.7	
	½	71.5		78.3			½	87.4		94.2	
	9/16	75.8		83.4			9/16	92.0		99.7	
	⅝	80.0	2.8	88.5	3.6		⅝	96.8	3.1	105.2	3.9
21.25	¼	59.5	2.7	62.9	3.3						
	5/16	63.7		68.0							
	⅜	68.0		73.1							
	7/16	72.3		78.2							
	½	76.5		83.3							
	9/16	80.8		88.4							
	⅝	85.0	2.8	93.5	3.6						

PLATE AND CHANNEL COLUMNS (Continued)

(Supplement to Table 18) TABLE 19

1	2	3	4	5	6	1	2	3	4	5	6
10″ CHANNELS						12″ CHANNELS					
		12″ Plates		15″ Plates				14″ Plates		16″ Plates	
Weight of each Channel	Thickness of Plates	Weight of Column	Least Radius of Gyration	Weight of Column	Radius of Gyration equal on both Axes	Weight of each Channel	Thickness of Plates	Weight of Column	Least Radius of Gyration	Weight of Column	Radius of Gyration equal on both Axes
Lbs. per ft.	Inches	Lbs. per ft.	Inches	Lbs. per ft.	Inches	Lbs. per ft.	Inches	Lbs. per ft	Inches	Lbs. per ft	Inches
15	¼	50.4	3.6	55.5	4.5	**20.5**	¼	64.8	4.4	68.2	5.2
	5/16	55.5		61.9			5/16	70.8		75.0	
	⅜	60.6		68.3			⅜	76.7		81.8	
	7/16	65.7		74.6			7/16	82.7		88.6	
	½	70.8		81.0			½	88.6		95.4	
	9/16	75.9		87.4			9/16	94.6		102.2	
	⅝	81.0	3.5	93.8	4.6		⅝	100.5	4.3	109.0	5.0
20	¼	60.4	3.5	65.5	4.3	**25**	¼	73.8	4.4	77.2	5.1
	5/16	65.5		71.9			5/16	79.8		84.0	
	⅜	70.6		78.3			⅜	85.7		90.8	
	7/16	75.7		84.6			7/16	91.7		97.6	
	½	80.8		91.0			½	97.6		104.4	
	9/16	85.9		97.4			9/16	103.6		111.2	
	⅝	90.1	3.5	103.8	4.6		⅝	109.5	4.3	118.0	5.0
25	¼	70.4	3.4	75.5	4.1	**30**	¼	83.8	4.3	87.2	4.9
	5/16	75.5		81.9			5/16	89.8		90.4	
	⅜	80.6		88.3			⅜	95.7		100.8	
	7/16	85.7		94.6			7/16	101.7		107.6	
	½	90.8		101.0			½	107.6		114.4	
	9/16	95.9		107.4			9/16	113.6		121.2	
	⅝	101.0	3.4	113.8	4.6		⅝	119.5	4.2	128.0	5.0
30	¼	80.4	3.3	85.5	4.0	**35**	¼	93.8	4.2	97.2	4.8
	5/16	85.5		91.9			5/16	99.8		104.0	
	⅜	90.6		98.3			⅜	105.7		110.8	
	7/16	95.7		104.6			7/16	111.7		117.6	
	½	100.8		111.0			½	117.6		124.4	
	9/16	105.9		117.4			9/16	123.6		131.2	
	⅝	111.0	3.4	123.8	4.5		⅝	129.5	4.1	138.0	4.9
35	¼	90.4	3.3	95.5	3.9	**40**	¼	103.8	4.1	107.2	4.7
	5/16	95.5		101.9			5/16	109.8		114.0	
	⅜	100.6		108.3			⅜	115.7		120.8	
	7/16	105.7		114.6			7/16	121.7		127.6	
	½	110.8		121.0			½	127.6		134.4	
	9/16	115.9		127.4			9/16	133.6		141.2	
	⅝	121.0	3.3	133.8	4.4		⅝	139.5	4.1	148.0	4.9

PLATE AND CHANNEL COLUMNS (Concluded)

(Supplement to Table 18)

TABLE 19

1	2	3	4	5	6	1	2	3	4	5	6	1	2	3	4	5	6
15″ CHANNELS																	
		17″ Plates		20′ Plates				17″ Plates		20″ Plates				17″ Plates		20′ Plates	
Weight of each Channel	Thickness of Plates	Weight of Column	Least Radius of Gyration	Weight of Column	Radius of Gyration equal on both Axes	Weight of each Channel	Thickness of Plates	Weight of Column	Least Radius of Gyration	Weight of Column	Radius of Gyration equal on both Axes	Weight of each Channel	Thickness of Plates	Weight of Column	Least Radius of Gyration	Weight of Column	Radius of Gyration equal on both Axes
Lbs. per ft.	Inches	Lbs. per ft.	Inches	Lbs. per ft.	Inches	Lbs. per ft.	Inches	Lbs. per ft.	Inches	Lbs. per ft.	Inches	Lbs. per ft.	Inches	Lbs. per ft.	Inches	Lbs. per ft.	Inches
33	3/8	109.4	5.4	117.0	6.6	**40**	3/8	123.4	5.4	131.0	6.4	**50**	3/8	143.4	5.3	151.0	6.2
	7/16	116.6		125.5			7/16	130.6		139.5			7/16	150.6		159.5	
	1/2	123.8		134.0			1/2	137.8		148.0			1/2	157.8		168.0	
	9/16	131.0		142.5			9/16	145.0		156.5			9/16	165.0		176.5	
	5/8	138.2		151.0			5/8	152.2		165.0			5/8	172.2		185.0	
	11/16	145.4		159.5			11/16	159.4		173.5			11/16	179.4		193.5	
	3/4	152.7	5.2	168.0	6.4		3/4	166.7	5.3	182.0	6.4		3/4	186.7	5.2	202.0	6.4
35	3/8	113.4	5.4	121.0	6.5	**45**	3/8	133.4	5.3	141.0	6.3	**55**	3/8	153.4	5.2	161.0	6.1
	7/16	120.6		129.5			7/16	140.6		149.5			7/16	160.6		169.5	
	1/2	127.8		138.0			1/2	147.8		158.0			1/2	167.8		178.0	
	9/16	135.0		146.5			9/16	155.0		166.5			9/16	175.0		186.5	
	5/8	142.2		155.0			5/8	162.2		175.0			5/8	182.2		195.0	
	11/16	149.4		163.5			11/16	169.4		183.5			11/16	189.4		203.5	
	3/4	156.7	5.3	172.0	6.4		3/4	176.7	5.2	192.0	6.4		3/4	196.7	5.1	212.0	6.3

STEEL ANGLES—(Even Legs)

For Beams, Girders, Columns, or Truss Members TABLE 20

1	2	3	4	5	6	7	8	9	10	11	12	13	14
		Weight per Foot				Neutral Axis on Line 45° to Legs		Neutral Axis Perpendicular to Leg					Gages
									Radius of Gyration				
Size	Thickness of Metal	One Angle	Two Angles	Four Angles	Area of Section of One Angle	Radius of Gyration	Distance from C. G. to Apex	Perpendicular Distance from C. G. to Back	Single Angle	Two Angles ½" Apart	Section-Moment (Single Angle)	Max. Size of Rivet	Single and Double Lines of Rivets in Leg
Inches	Ins.	Lbs.	Lbs.	Lbs.	Square Inches	Ins.	Ins.	Ins.	Ins.	Ins.	Foot-Lbs.	In.	Inches.
1 x1	⅛	0.8	1.6	3.2	0.24	0.20	0.42	0.30	0.31	0.62	41	⅜	9/16
	3/16	1.2	2.3	4.7	0.34						58		
	¼	1.5	3.0	6.0	0.44	0.19	0.48	0.34	0.29	0.66	74		
1¼x1¼	⅛	1.0	2.0	4.1	0.30	0.25	0.50	0.35	0.38	0.72	65	½	¾
	3/16	1.5	3.0	5.9	0.43						94		
	¼	1.9	3.8	7.6	0.56						121		
	5/16	2.4	4.7	9.3	0.68	0.23	0.60	0.42	0.36	0.77	145		
1½x1½	⅛	1.2	2.5	4.9	0.36	0.30	0.60	0.42	0.46	0.83	93	½	⅞
	3/16	1.8	3.6	7.2	0.53						138		
	¼	2.4	4.7	9.4	0.69						178		
	5/16	2.9	5.7	11.4	0.84						216		
	⅜	3.4	6.7	13.4	0.99	0.29	0.72	0.51	0.44	0.88	253		
1¾x1¾	3/16	2.1	4.2	8.4	0.62	0.35	0.72	0.51	0.54	0.93	186	⅝	1
	¼	2.8	5.5	11.0	0.81						253		
	5/16	3.4	6.8	13.6	1.00						306		
	⅜	4.0	8.0	15.9	1.17						346		
	7/16	4.6	9.2	18.3	1.30	0.33	0.83	0.59	0.51	0.98	400		
2 x2	3/16	2.5	5.0	10.0	0.72	0.40	0.80	0.57	0.62	1.03	253	⅝	1⅛
	¼	3.2	6 4	12.8	0.94						333		
	5/16	4.0	8 0	16.0	1.15						400		
	⅜	4.7	9.4	18.8	1.36						467		
	7/16	5 3	10.6	21.2	1.56	0.39	0.93	0.66	0.59	1.08	533		
2¼x2¼ Special	3/16	2.8	5.6	11.2	0.81	0.44	0.89	0.63	0.70	1.12	320	¾	1¼
	¼	3.7	7.4	14.8	1.06						426		
	5/16	4.5	9.0	18.0	1.31						520		
	⅜	5.3	10.6	21.2	1.55						600		
	7/16	6.1	12.2	24.4	1.78						693		
	½	6.8	13.6	27.2	2.00	0.43	1.05	0.74	0.66	1.19	773		
2½x2½	3/16	3.1	6.2	12.4	0 90	0.49	0.98	0.69	0.78	1.22	400	¾	1⅜
	¼	4.1	8.2	16.4	1.19						533		
	5/16	5.0	10.0	20.0	1.47						640		
	⅜	5.9	11.8	23.6	1.73						760		
	7/16	6.8	13.6	27.2	2.00						866		
	½	7.7	15.4	30.8	2.25	0.47	1.15	0.81	0.74	1.29	973		
2¾x2¾ Special	¼	4.5	9.0	18.0	1.31	0.55	1.10	0.78	0.85	1.34	640	¾	1⅝
	5/16	5.5	11.0	22.0	1.62						786		
	⅜	6.6	13.2	26.4	1.92						920		
	7/16	7.6	15.2	30.4	2.22						1050		
	½	8.5	17.0	34.0	2.50	0.52	1.23	0.87	0.82	1.39	1180		
3 x3	¼	4.9	9.8	19.6	1.44	0.59	1.19	0.84	0.93	1.43	773	⅞	1¾
	5/16	6.1	12.2	24.4	1.78						946		
	⅜	7.2	14.4	28.8	2.11						1100		
	7/16	8.3	16.6	33.2	2.43						1260		
	½	9.4	18.8	37.6	2.75						1420		
	9/16	10.4	20.8	41.6	3.06						1580		
	⅝	11.4	22.8	45.6	3.36	0.57	1.41	1.00	0.88	1.51	1730		

Above angles are rolled by nearly all mills.

STEEL ANGLES—(Even Legs)—Continued.

For Beams, Girders, Columns, or Truss Members TABLE 20

1	2	3	4	5	6	7	8	9	10	11	12	13	14
Size	Thickness of Metal	Weight per Foot			Area of Section of One Angle	Neutral Axis on Line 45° to Legs		Neutral Axis Perpendicular to Leg.				Max. Size of Rivet	Gages
		One Angle	Two Angles	Four Angles		Radius of Gyration	Distance from C. G. to Apex	Perpendicular Distance from C. G. to Back	Radius of Gyration: Single Angle	Radius of Gyration: Two Angles ½" Apart	Section-Moment (Single Angle)		Single and Double Lines of Rivets in Leg
Inches	Ins.	Lbs.	Lbs.	Lbs.	Square Inches	Ins.	Ins.	Ins.	Ins.	Ins.	Foot-Lbs.	In.	Inches.
3½x3½	5/16	7.1	14.2	28.4	2.09	0.69	1.4	1.0	1.08	1.65	1300	⅞	2
	⅜	8.5	17.0	34.0	2.48						1530		
	7/16	9.8	19.6	39.2	2.87						1760		
	½	11.1	22.2	44.4	3.25						1980		
	9/16	12.3	24.6	49.2	3.62						2200		
	⅝	13.6	27.2	54.4	3.98						2410		
	11/16	14 8	29.6	59.2	4.34						2610		
	¾	16.0	32.0	64.0	4.69						2810		
	13/16	17.1	34.2	68.4	5.03	0.67	1.6	1.2	1.02	1.74	3000		
4 x4	5/16	8.2	16.4	32.8	2.40	0.79	1.6	1.1	1.24	1.85	1720	⅞	2¼
	⅜	9.8	19.6	39.2	2.86						2020		1¾-1
	7/16	11.3	22.6	45.2	3.31						2330		
	½	12.8	25.6	51.2	3.75						2620		
	9/16	14 3	28.6	57.2	4.18						2920		
	⅝	15.7	31.4	62 8	4.61						3200		
	11/16	17.1	34.2	68.4	5.03						3480		
	¾	18.5	37.0	74.0	5.44						3740		
	13/16	19.9	39.8	79.6	5.84	0.77	1.8	1.3	1.18	1.94	4010		
5 x5 Special	⅜	12 3	24.6	49.2	3.61	0.99	2.0	1.4	1.56	2.27	3220	⅞	2¾
	7/16	14.3	28.6	57.2	4.18						3720		1¾-2
	½	16.2	32.4	64.8	4.75						4200		
	9/16	18.1	36.2	72.4	5.31						4690		
	⅝	20.0	40.0	80.0	5.86						5150		
	11/16	21.8	43.6	87.2	6.42						5600		
	¾	23.6	47.2	94.4	6.94						6050		
	13/16	25.4	50.8	101.6	7.46						6470		
	⅞	27.2	54.4	108.8	7.99						6900		
	15/16	28.9	57.8	115.6	8.50						7320		
	1	30.6	61.2	122.4	9.00	0.96	2.3	1.6	1.48	2.38	7730		
6 x6	⅜	14.8	29.6	59.2	4.36	1.19	2.3	1.6	1.88	2.66	4710	⅞	3½
	7/16	17.2	34.4	68.8	5.06						5430		2¼-2½
	½	19.6	39.2	78.4	5.75						6140		
	9/16	21.9	43.8	87.6	6.43						6850		
	⅝	24.2	48.4	96.8	7.11						7550		
	11/16	26.5	53.0	106.0	7.78						8230		
	¾	28.7	57.4	114.8	8.44						8900		
	13/16	30.9	61.8	123.6	9.09						9530		
	⅞	33.1	66.2	132.4	9.74						10190		
	15/16	35.3	70.6	141.2	10 37						10810		
	1	37.4	74.8	149.6	11.00	1.16	2.6	1.8	1.80	2.77	11570		
8 x8 Rolled by Carnegie and Pencoyd only.	½	26.4	52.8	105 6	7.75	1.58	3.1	2.2	2.50	3.49	11170	⅞	4½
	9/16	29.5	59.0	118.0	8.68						12450		2¾-3¼
	⅝	32.7	65.4	130.8	9.61						13720		
	11/16	35.8	71.6	143.2	10.53						15000		
	¾	38.9	77.8	155 6	11.44						16210		
	13/16	42.0	84.0	168.0	12 34						17500		
	⅞	45.0	90.0	180.0	13 23						18700		
	15/16	48.0	96.0	192.0	14.12						19900		
	1	51.0	102.0	204.0	15.00						21060		
	1 1/16	54.0	108.0	216.0	15.87						22200		
	1⅛	56.9	113.8	227.6	16.73	1.55	3.4	2.4	2.42	3.60	23400		

Above angles are rolled by nearly all mills.

STEEL ANGLES—(Uneven Legs)

For Beams, Girders, Columns, or Truss Members TABLE 21

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
		Weight per Foot					Long Leg Perpendicular to Neutral Axis				Short Leg Perpendicular to Neutral Axis			
								Radius of Gyration				Radius of Gyration		
Size	Thickness of Metal	One Angle	Two Angles	Four Angles	Area of Section of One Angle	Least Radius of Gyration of Single Angle	Perpendicular Distance from C. G. to Back	Single Angle	Two Angles ½" Apart	Section-Moment (Single Angle)	Perpendicular Distance from C. G. to Back	Single Angle	Two Angles ½" Apart	Section-Moment (Single Angle)
Inches	Ins.	Lbs.	Lbs.	Lbs.	Sq. Ins.	Ins.	Ins.	Ins.	Ins.	Foot-Lbs.	Ins.	Ins.	Ins.	Foot-Lbs.
1⅜x1 Special	⅛	1.0	2.0	4.0	0.28	0.22	0.44	0.44		80	0.26	0.29		40
	¼	1.8	3.6	7.2	0.53		0.48			120	0.29			66
2 x1⅜ Special	3/16	2.1	4.2	8.4	0.60	0.31	0.66	0.63		240	0.35	0.40		120
	¼	2.7	5.4	10.8	0.78		0.69			306	0.37			160
2¼x1½ Special	3/16	2.3	4.6	9.2	0.67	0.40	0.75	0.72		306	0.37	0.43		146
	¼	3.0	6.0	12.0	0.88					400				186
	5/16	3.7	7.4	14.8	1.07					480				226
	⅜	4.3	8.6	17.2	1.27					560				266
	7/16	5.0	10.0	20.0	1.45					640				307
	½	5.5	11.0	22.0	1.63	0.39	0.86	0.71		786	0.48	0.40		346
2½x2	3/16	2.8	5.6	11.2	0.81	0.43	0.76	0.79	1.29	386	0.51	0.60	0.97	266
	¼	3.7	7.4	14.8	1.06					506				333
	5/16	4.5	9.0	18.0	1.31					626				413
	⅜	5.3	10.6	21.2	1.55					733				480
	7/16	6.1	12.2	24.4	1.78					826				546
	½	6.8	13.6	27.2	2.00	0.42	0.88	0.75	1.35	933	0.63	0.56	1.04	613
3 x2 Special	¼	4.0	8.0	16.0	1.19	0.43	0.99	0.95	1.56	720	0.49	0.57	0.93	333
	5/16	5.0	10.0	20.0	1.47					780				426
	⅜	5.9	11.8	23.6	1.73					1040				493
	7/16	6.8	13.6	27.2	2.00					1180				560
	½	7.7	15.4	30.8	2.25	0.43	1.08	0.92	1.61	1330	0.58	0.55	0.98	626
3 x2½	¼	4.5	9.0	18.0	1.31	0.53	0.91	0.95	1.50	746	0.66	0.75	1.18	533
	5/16	5.5	11.0	22.0	1.62					920				653
	⅜	6.6	13.2	26.4	1.92					1080				773
	7/16	7.6	15.2	30.4	2.22					1240				880
	½	8.5	17.0	34.0	2.50					1380				986
	9/16	9.5	19.0	38.0	2.78	0.52	1.02	0.91	1.56	1530	0.77	0.72	1.25	1090
3¼x2 Special	¼	4.3	8.6	17.2	1.25	0.45	1.09	1.04	1.70	840	0.48	0.57	0.92	346
	5/16	5.3	10.6	21.2	1.54					1020				426
	⅜	6.2	12.4	24.8	1.83					1210				493
	7/16	7.2	14.4	28.8	2.11					1400				573
	½	8.1	16.2	32.4	2.38					1560				640
	9/16	9.0	18.0	36.0	2.64	0.44	1.21	1.00	1.77	1730	0.59	0.53	0.99	706
3½x2½	¼	4.9	9.8	19.6	1.44	0.54	1.11	1.12	1.76	1000	0.61	0.74	1.13	546
	5/16	6.1	12.2	24.4	1.78					1240				666
	⅜	7.2	14.4	28.8	2.11					1450				786
	9/16	8.3	16.6	33.2	2.43					1680				906
	½	9.4	18.8	37.6	2.75					1880				1010
	7/16	10.4	20.8	41.6	3.06					2080				1120
	⅝	11.4	22.8	45.6	3.36					2280				1220
	11/16	12.4	24.8	49.6	3.65	0.53	1.27	1.06	1.86	2460	0.77	0.67	1.23	1320

Above angles are rolled by nearly all mills.

STEEL ANGLES—(Uneven Legs)—Continued

For Beams, Girders, Columns, or Truss Members TABLE 21

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
Size	Thickness of Metal	Weight per Foot			Area of Section of One Angle	Least Radius of Gyration of Single Angle	Long Leg Perpendicular to Neutral Axis				Short Leg Perpendicular to Neutral Axis			
		One Angle	Two Angles	Four Angles			Perpendicular Distance from C. G. to Back	Radius of Gyration		Section-Moment (Single Angle)	Perpendicular Distance from C. G. to Back	Radius of Gyration		Section-Moment (Single Angle)
								Single Angle	Two Angles ½" Apart			Single Angle	Two Angles ½" Apart	
Inches	Ins.	Lbs.	Lbs.	Lbs.	Sq. Ins.	Ins.	Ins.	Ins.	Ins.	Foot-Lbs.	Ins.	Ins.	Ins.	Foot-Lbs.
3½x3	5/16	6.6	13.2	26.4	1.93	0.63	1.06	1.10	1.71	1280	0.81	0.90	1.39	960
	3/8	7.8	15.6	31.2	2.30					1510				1130
	7/16	9.1	18.2	36.4	2.65					1720				1300
	1/2	10.2	20.4	40.8	3.00					1930				1460
	9/16	11.4	22.8	45.6	3.34					2140				1610
	5/8	12.5	25.0	50.0	3.67					2340				1770
	11/16	13.6	27.2	54.4	4.00					2530				1920
	3/4	14.7	29.4	58.8	4.31					2730				2050
	13/16	15.7	31.4	62.8	4.62	0.62	1.23	1.04	1.81	2930	0.98	0.85	1.50	2200
4 x3	5/16	7.1	14.2	28.4	2.09	0.65	1.26	1.27	1.97	1640	0.76	0.89	1.34	985
	3/8	8.5	17.0	34.0	2.48					1940				1160
	7/16	9.8	19.6	39.2	2.87					2240				1320
	1/2	11.1	22.2	44.4	3.25					2520				1490
	9/16	12.3	24.6	49.2	3.62					2780				1640
	5/8	13.6	27.2	54.4	3.98					3070				1800
	11/16	14.8	29.6	59.2	4.34					3320				1940
	3/4	16.0	32.0	64.0	4.69					3570				2090
	13/16	17.1	34.2	68.4	5.03	0.64	1.44	1.21	2.08	3830	0.94	0.83	1.45	2240
4 x3½ Special	5/16	7.7	15.4	30.8	2.25	0.73	1.18	1.26	1.91	1680	0.93	1.07	1.59	1340
	3/8	9.1	18.2	36.4	2.67					2000				1570
	7/16	10.5	21.0	42.0	3.09					2390				1800
	1/2	11.9	23.8	47.6	3.50					2570				2020
	9/16	13.3	26.6	53.2	3.90					2860				2240
	5/8	14.6	29.2	58.4	4.30					3130				2450
	11/16	15.9	31.8	63.6	4.68					3410				2660
	3/4	17.2	34.4	68.8	5.06					3670				2860
	13/16	18.5	37.0	74.0	5.43	0.72	1.36	1.19	2.01	3890	1.11	1.01	1.69	3320
4½x3 Special	5/16	7.7	15.4	30.8	2.25	0.66	1.47	1.44	2.26	2050	0.72	0.88	1.31	1010
	3/8	9.1	18.2	36.4	2.67					2440				1170
	7/16	10.5	21.0	42.0	3.09					2800				1340
	1/2	11.9	23.8	47.6	3.50					3160				1500
	9/16	13.3	26.6	53.2	3.90					3520				1660
	5/8	14.6	29.2	58.4	4.30					3850				1820
	11/16	15.9	31.8	63.6	4.68					4180				1980
	3/4	17.2	34.4	68.8	5.06					4510				2130
	13/16	18.5	37.0	74.0	5.43	0.64	1.65	1 38	2.35	4820	0.90	0.81	1.46	2280
5 x3	5/16	8.2	16.4	32.8	2.40	0.66	1.68	1.61	2.52	2520	0.68	0.85	1.26	1000
	3/8	9.8	19.6	39.2	2.86					2980				1180
	7/16	11.3	22.6	45.2	3.31					3440				1360
	1/2	12.8	25.6	51.2	3.75					3880				1530
	9/16	14.2	28.4	56.8	4.18					4310				1690
	5/8	15.7	31.4	62 8	4.61					4730				1850
	11/16	17.1	34.2	68.4	5.03					5150				2010
	3/4	18.5	37.0	74.0	5.44					5550				2170
	13/16	19.9	39.8	79.6	5.84	0.64	1.86	1.55	2.62	5940	0.86	0.80	1.37	2320

Above angles are rolled by nearly all mills.

STEEL ANGLES—(Uneven Legs)—Continued

For Beams, Girders, Columns, or Truss Members TABLE 21

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
Size	Thickness of Metal	Weight per Foot			Area of Section of One Angle	Least Radius of Gyration of Single Angle	Long Leg Perpendicular to Neutral Axis				Short Leg Perpendicular to Neutral Axis			
		One Angle	Two Angles	Four Angles			Perpendicular Distance from C. G. to Back	Radius of Gyration		Section-Moment (Single Angle)	Perpendicular Distance from C. G. to Back	Radius of Gyration		Section-Moment (Single Angle)
								Single Angle	Two Angles ½" Apart			Single Angle	Two Angles ½" Apart	
Inches	Ins.	Lbs.	Lbs.	Lbs.	Sq. Ins.	Ins.	Ins.	Ins.	Ins.	Foot-Lbs.	Ins.	Ins.	Ins.	Foot-Lbs.
5 x3½	5/16	8.7	17.4	34.8	2.56	0.76	1.59	1.61	2.45	2580	0.84	0.03	1.51	1360
	3/8	10.4	20.8	41.6	3 05					3050				1610
	7/16	12.0	24.0	48.0	3.53					3250				1850
	½	13.6	27.2	54.4	4.00					3990				2080
	9/16	15.2	30.4	60.8	4.47					4430				2300
	5/8	16.8	33.6	67.2	4.92					4860				2530
	11/16	18.3	36.6	73.2	5.37					5290				2750
	¾	19.8	39.6	79.2	5.81					5710				2960
	13/16	21.3	42.6	85.2	6.25					6110				3160
	7/8	22.7	45.4	90.8	6.67	0.75	1.79	1.53	2.55	7510	1.04	0.96	1.61	3360
6 x3½	3/8	11.7	23.4	46.8	3.42	0.77	2.04	1.94	3.00	4330	0.79	0.99	1.43	1640
	7/16	13 5	27.0	54 0	3.97					5000				1880
	½	15.3	30.6	61.2	4.59					5650				2120
	9/16	17.1	34.2	68.4	5.03					6300				2360
	5/8	18.9	37.8	75.6	5.55					6920				2580
	11/16	20.6	41.2	82.4	6.06					7530				2810
	¾	22.3	44.6	89.2	6.56					8139				3030
	13/16	24.0	48.0	96.0	7.06					8730				3240
	7/8	25.7	51 4	102.8	7 55					9300				3450
	15/16	27.3	54.6	109.2	8.03					9890				3650
	1	28.9	57.8	115.6	8.50	0.74	2.26	1.85	3.10	10420	1.01	0.92	1.56	3860
6 x4	3/8	12.3	24.6	49.2	2.61	0 88	1.94	1.93	2.92	4420	0.94	1.17	1.67	2130
	7/16	14.3	28.6	57.2	4.18					5110				2460
	½	16.2	32.4	64 8	4 75					5780				2870
	9/16	18.1	36.2	72 4	5.31					6440				3080
	5/8	20.0	40.0	80.0	5.86					7080				3480
	11/16	21.8	43.6	87.2	6.41					7710				3680
	¾	23.6	47.2	94.4	6.94					8330				3960
	13/16	25.4	50.8	101.6	7.47					8930				4240
	7/8	27.2	54.4	108.8	7.99					9530				4530
	15/16	28.9	57.8	115.6	8.50					10110				4789
	1	30.6	61.2	122.4	9.00	0.85	2.17	1.85	3.02	10700	1.17	1.09	1.79	5050
7 x3½	7/16	15.0	30.0	60.0	4.40	0.89	2.50	2.26	3.56	6680	0.75	0.95	1.38	1960
Special.	½	17.0	34.0	68.0	5.00					7570				2160
Rolled	9/16	19.0	38.0	76 0	5.59					8440				2400
by	5/8	21.0	42.0	84.0	6.17					9300				2630
Carnegie	11/16	23.0	46.0	92.0	6.75					10130				2850
and	¾	24.9	49.8	99.6	7.31					10950				3080
Pencoyd	13/16	26.8	53 6	107.2	7.87					11750				3310
only	7/8	28.7	57.4	114.8	8.42					12570				3520
	15/16	30.5	60 1	120.2	8.97					13350				3730
	1	32.3	64.6	129.2	9.50	0.88	2.71	2.10	3.68	14100	0.96	0.89	1.50	3950
8 x6	½	23.0	46.0	92.0	6.76	1.34	2.47	2.56	3.74	10710	1.47	1.79	2.48	6410
Pencoyd	9/16	25.8	51.6	103.2	7.59					11946				7142
only	5/8	28.7	57.4	114.8	8.44					13180				7875
	11/16	31.7	63.4	126.8	9.32					14420				8607
	¾	33.8	67.6	135.2	9.94					15655				9340
	13/16	36.6	73.2	146.4	10.76					16890				10070
	7/8	39.5	79 0	158.0	11.62					18127				10805
	15/16	42.5	85.0	170.0	12.50					19363				11537
	1	45.6	91.2	182.4	13.41	1.37	2.72	2.53	3.90	20600	1.72	1.77	2.65	12270

Above angles are rolled by all mills except as noted.

PLATE AND ANGLE COLUMNS.—(Supplement to Table 21.)

TABLE 22

WITHOUT COVER PLATES

Weight per Lineal Foot of Cross Section Including Angles and Web Plate

Size of Angles	Thickness	⅜″ Web Plate 6″ Pl.	⅜″ Web Plate 8″ Pl.	⅜″ Web Plate 10″ Pl.
		Lbs. per L. ft.	Lbs. per L. ft.	Lbs. per L. ft.
3x2½	¼	25.7	28.2	30.8
	5/16	29.7	32.2	34.8
	⅜	34.1	36.6	39.2
	7/16	38.1	40.6	43.2
	½	41.7	44.2	46.8
3x3	¼		29.8	32.4
	5/16		34.6	37.2
	⅜		39.0	41.6
	7/16		43.4	46.0
	½		47.8	50.4
3½x3	5/16		36.6	39.2
	⅜		41.4	44.0
	7/16		46.6	49.2
	½		51.0	53.6
	9/16		55.8	58.4
	⅝		60.2	62.8
4x3	5/16		38.6	41.2
	⅜		44.2	46.8
	7/16		49.6	52.0
	½		54.6	57.2
	9/16		59.4	62.0
	⅝		64.6	67.2

Size of Angles	Thickness	½″ Web Plate 10″ Pl.	½″ Web Plate 12″ Pl.	½″ Web Plate 14″ Pl.	⅝″ Web Plate 12″ Pl.	⅝″ Web Plate 14″ Pl.	⅝″ Web Plate 16″ Pl.
		Lbs. per L. ft.	Lbs. per L. ft.	Lbs. per L. ft.	Lbs. per L. ft.	Lbs. per L. ft.	Lbs. per L. ft.
5x3	5/16	49.8	53.2	56.6	58.3	62.6	
	⅜	56.2	59.6	63.0	64.7	69.0	
	7/16	62.2	65.6	69.0	70.7	75.0	
	½	68.2	71.6	75.0	76.7	81.0	
	9/16	73.8	77.2	80.6	82.3	86.6	
	⅝	79.8	83.2	86.6	88.3	92.6	
6x3½	⅜	63.8	67.2	70.6	72.3	76.6	
	7/16	71.0	74.4	77.8	79.5	83.8	
	½	78.2	81.6	85.0	86.7	91.0	
	9/16	85.4	88.8	92.2	93.9	98.2	
	⅝	92.6	96.0	99.4	101.1	105.4	
6x4	⅜		69.6	73.0	74.7	79.0	83.2
	7/16		77.6	81.0	82.7	87.0	91.2
	½		85.2	88.6	90.3	94.6	98.8
	9/16		92.8	96.2	97.9	102.2	106.4
	⅝		100.4	103.8	105.5	109.8	114.0
	11/16		107.6	111.0	112.7	117.0	121.2
	¾		114.8	118.2	119.9	124.2	128.4
	13/16		122.0	125.4	127.1	131.4	135.6
	⅞		129.2	132.6	134.3	138.6	142.8

WITH COVER PLATES

Weight per Lineal Foot of Cross Section Including Angles, Web and Cover Plates

Combined Thickness of Cover Plates	4 Ls 6″x4″x½″ Web Pl. 12″x⅝″ 13″ Cov. Pls.	4 Ls 6″x4″x¾″ Web. Pl. 14″x¾″ 13″ Cov. Pls.	4 Ls 6″x4″x¾″ Web. Pl. 14″x¾″ 15″ Cov. Pls.
	Lbs. per L. ft.	Lbs. per L. ft.	Lbs. per L. ft.
¾	123.5	163.3	168.4
1	134.5	174.3	181.1
1¼	145.6	185.4	193.9
1½	156.6	196.4	206.6
1¾	167.7	207.5	219.4
2	178.8	218.5	232.1
2¼	189.8	229.6	244.9
2½	200.9	240.6	257.6
2¾	211.9	251.7	270.4
3	223.0	262.7	283.1

STEEL TEES

For Beams, Girders, Columns, or Truss Members

TABLE 23

1	2	3	4	5	6	7	8	9
Dimensions and Weights					Axis Parallel with Flange		Axis Coincident with Stem	
Flange	Stem	Weight per foot	Area	Distance of Center of Gravity from outside of Flange	Radius of Gyration	Section-Moment	Radius of Gyration	Section-Moment
Ins.	Ins.	Lbs.	Sq. Ins.	Ins.	Ins.	Foot-lbs.	Ins.	Foot-lbs.
1	1	0.87	0 26	0 29	0.29	40	0.21	26
	1	1.23	0.36	0.32	0.29	66	0.21	53
1¼	1¼	1.53	0 45	0.38	0 37	93	0.26	66
	1¼	2.04	0.60	0.40	0 36	133	0 27	93
1½	1½	1.84	0.54	0.44	0.45	146	0 31	93
	1½	2.4	0 75	0.42	0.49	186	0 34	133
1¾	1¼	3.6	1.05	0.91	0.33	200	0 41	293
	1¾	3.1	0.90	0.54	0.51	253	0.37	186
2	1½	3.1	0.90	0.42	0.42	200	0.45	240
	2	3.7	1.08	0 59	0.60	330	0 42	240
	2	4.3	1.26	0.63	0.60	440	0.43	306
2¼	2¼	4.1	1.20	0 66	0 67	426	0.47	293
	2¼	4.9	1.44	0.69	0.68	560	0.48	400
2½	1¼	2.9	0.84	0.29	0 31	120	0 58	306
	2½	5.5	1.62	0.74	0.74	666	0 52	466
	2½	6 4	1.89	0 76	0.74	786	0.53	560
	2¾	5 8	1.71	0 83	0 83	800	0 51	466
	2¾	6 7	1 98	0 87	0 84	972	0.58	706
	3	6.1	1.80	0 92	0.94	1010	0.51	466
	3	7.2	2.10	0.97	0.92	1160	0.51	573
2¾	2	7.4	2.16	0.53	0.71	1000	0.54	600
3	2½	6 1	1 80	0.68	0.73	693	0.65	666
	2½	7.2	2.10	0 71	0 72	800	0 66	800
	3	6.6	1 95	0.86	0.90	985	0 62	666
	3	7 8	2.28	0.88	0 90	1140	0 63	800
	3	9.1	2.67	0 92	0 90	1340	0 64	960
	3	10.0	2 94	0.93	0.88	1460	0.64	1070
	3½	8 5	2.49	1.09	1.09	1610	0.61	826
	3½	9.8	2.88	1.11	1.08	1825	0.68	1170
	3½	10.9	3.21	1.12	1.06	1980	0.62	1070
	4	9.3	2.73	1.29	1.26	2090	0.59	826
	4	10.6	3.12	1.32	1.25	2370	0.60	960
	4	11.8	3.48	1.32	1.23	2580	0.59	1080

As there is no uniform standard for Tees, the Carnegie rolls are given as representative for variety of size and weight.

STEEL TEES—Continued

For Beams, Girders, Columns, or Truss Members

TABLE 28

1	2	3	4	5	6	7	8	9
Dimensions and Weights					Axis Parallel with Flange		Axis Coincident with Stem	
Flange	Stem	Weight per foot	Area	Distance of Center of Gravity from outside of Flange	Radius of Gyration	Section-Moment	Radius of Gyration	Section-Moment
Ins.	Ins.	Lbs.	Sq. Ins.	Ins.	Ins.	Foot-lbs.	Ins.	Foot-lbs.
3½	3	7.8	2.28	0.78	0.89	960	0.76	906
	3	8.5	2.49	0.83	0.88	1170	0.75	1080
	3	10.9	3.21	0.88	0.87	1500	0.77	1440
	3½	9.2	2.70	1.01	1.05	1590	0.73	1080
	3½	11.7	3.45	1.06	1.04	2025	0.74	1440
	4	9.9	2.91	1.19	1.22	2060	0.70	1080
	4	12.8	3.75	1.25	1.21	2640	0.72	1440
4	2	6.6	1.95	0.51	0.51	454	0.95	1170
	2	7.9	2.31	0.48	0.52	534	0.96	1400
	2½	7.3	2.16	0.60	0.70	733	0.91	1170
	2½	8.6	2.52	0.63	0.69	826	0.92	1400
	3	9.3	2.73	0.78	0.86	1170	0.88	1400
	4	10.9	3.21	1.15	1.23	2180	0.84	1454
	4	13.7	4.02	1.18	1.20	2690	0.84	1870
	4½	11.4	3.36	1.31	1.38	2640	0.80	1410
	4½	14.6	4.29	1.37	1.37	3400	0.81	1880
	5	12.0	3.54	1.51	1.56	3740	0.78	1410
	5	15.6	4.56	1.56	1.54	4140	0.79	1880
4½	2½	9.3	2.79	0.60	0.68	866	1.08	1840
	2½	8.0	2.40	0.58	0.69	746	1.07	1546
	3	10.0	3.00	0.75	0.86	1250	1.04	1840
	3	8.5	2.55	0.73	0.87	1080	1.03	1546
	3½	15.8	4.65	1.11	1.04	2840	0.90	2200
5	2½	11.0	3.24	0.65	0.71	1140	1.16	2260
	3	13.6	3.99	0.75	0.82	1570	1.19	2960
*	4	15.3	4.54	1.08	1.17	2810	1.09	2880
*	3½	17.0	4.95	1.06	1.03	2890	1.05	2920
* 6	5¼	39.0	11.58	1.75	1.57	10920	1.27	8330
*	4	15.6	4.61	0.97	1.12	2560	1.33	3140

*Pencoyd only.

STEEL ZEE-BARS

For Beams, Girders, Columns, or Truss Members TABLE 24

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
Dimensions			Weight per foot of one Zee-Bar	Beams	Zee-Bar Columns						Practical Detailing Dimensions			
							Radius of Gyration				Web		Flange	
Web Height	Flange Widths	Thickness of Metal		Section-Moment Axis Perpendicular to Web	Width of Web Plate	Weight per foot of 4 Z's and Web Plate	Axis Coincident with Web Plate	Axis Perpendicular to Web	Weight per foot of 4 Z's and Web Plate 8 inches wide	Radius of Gyration Axis Perpendicular to 8" Web Plate	Gage	Max. Size of Rivets	Gage	Max. Size of Rivets
In.	In.	In.	Lbs.	Foot-Tons	In.	Lbs.	In.	In.	Lbs.	Ins.	In.	In.	In.	In.
3	2 11/16	1/4	6.7	1.28	6	31.9	1.9	3.0	33.6	4.0	1 1/2	3/4	1 5/8	3/4
3 1/16	2 3/4	5/16	8.4	1.58		40.0			42.1					
3	2 11/16	3/8	9.7	1.72		46.5			49.0					
3 1/16	2 3/4	7/16	11.4	1.98		54.5			57.5					
3	2 11/16	1/2	12.5	2.04		60.2			63.6					
3 1/16	2 3/4	9/16	14.2	2.28		68.3	1.9	2.9	72.1	3.8				
4	3 1/16	1/4	8.2	2.09	6 1/2	38.3	2.5	3.3	39.6	4.0	2	7/8	2	3/4
4 1/16	3 1/8	5/16	10.3	2.60		48.1			49.7					
4 1/8	3 8/16	3/8	12.4	3.11		57.9			59.8					
4	3 1/16	7/16	13.8	3.22		64.9			67.1					
4 1/16	3 1/8	1/2	15.8	3.67		74.2			76.8					
4 1/8	3 8/16	9/16	17.9	4.12		84.0			86.9					
4	3 1/16	5/8	18.9	4.03		89.4			92.6					
4 1/16	3 1/8	11/16	20.9	4.43		98.8			102.3					
4 1/8	3 8/16	3/4	22.9	4.84		108.2	2.6	3.1	112.0	3.8				
5	3 1/4	5/16	11.6	3.56	7	53.8	3.1	3.5	54.9	4.0	2 1/2	7/8	2 1/8	7/8
5 1/16	3 5/16	3/8	13.9	4.26		64.5			65.8					
5 1/8	3 3/8	7/16	16.4	4.96		76.0			77.5					
5	3 1/4	1/2	17.8	5.12		83.1			84.8					
5 1/16	3 5/16	9/16	20.2	5.75		94.2			96.1					
5 1/8	3 3/8	5/8	22.6	6.38		105.3			107.4					
5	3 1/4	11/16	23.7	6.32		111.2			113.5					
5 1/16	3 5/16	3/4	26.0	6.89		121.9			124.4					
5 1/8	3 3/8	13/16	28.3	7.47		132.5	3.2	3.3	135.3	3.8				
6	3 1/2	3/8	15.6	5.63	7 1/2	72.0	3.7	3.7	72.6	4.0	3	7/8	2 1/4	7/8
6 1/16	3 9/16	7/16	18.3	6.55		84.4			85.1					
6 1/8	3 5/8	1/2	21.0	7.48		96.8			97.6					
6	3 1/2	9/16	22.7	7.70		105.1			106.1					
6 1/16	3 9/16	5/8	25.4	8.55		117.5			118.6					
6 1/8	3 5/8	11/16	28.0	9.40		129.5			130.7					
6	3 1/2	3/4	29.3	9.36		136.3			137.6					
6 1/16	3 9/16	13/16	32.0	10.15		148.7			150.1					
6 1/8	3 5/8	7/8	34.6	10.93		160.7	3.8	3.5	162.2	3.7				

WEIGHTS OF FLAT ROLLED STEEL

Pounds Per Lineal Foot

TABLE 25

Thickness of Metal	Width of Plates																			Thickness of Metal	Section for Rivet Holes Lbs. per L. ft. Diameter of Rivets	
Ins.	5"	5½"	6"	6½"	7"	7½"	8"	8½"	9"	9⅛"	10"	11"	12"	13"	14"	15"	16"	17"	18"	Ins.	½"0	¾"0
¼	4.25	4.67	5.10	5.53	5.95	6.36	6.80	7.22	7.65	8.08	8.50	9.34	10.20	11.06	11.90	12.75	13.60	14.44	15.30	¼	0.5	0.8
5/16	5.31	5.84	6.38	6.90	7.44	7.97	8.50	9.03	9.56	10.10	10.62	11.68	12.75	13.81	14.88	15.94	17.00	18.06	19.12	5/16	0.6	1.0
⅜	6.38	7.02	7.65	8.29	8.93	9.57	10.20	10.84	11.48	12.12	12.75	14.03	15.30	16.58	17.86	19.14	20.40	21.68	22.96	⅜	0.8	1.2
7/16	7.44	8.18	8.93	9.67	10.41	11.16	11.90	12.64	13.40	14.14	14.88	16.36	17.85	19.34	20.82	22.32	23.80	25.28	26.79	7/16	0.9	1.4
½	8.50	9.35	10.20	11.05	11.90	12.75	13.60	14.44	15.30	16.16	17.00	18.70	20.40	22.10	23.80	25.50	27.20	28.89	30.60	½	1.0	1.6
9/16	9.57	10.52	11.48	12.43	13.39	14.34	15.30	16.26	17.22	18.18	19.14	21.02	22.95	24.86	26.78	28.70	30.60	32.52	34.44	9/16	1.2	1.9
⅝	10.63	11.69	12.75	13.81	14.87	15.94	17.00	18.06	19.13	20.19	21.25	23.38	25.50	27.62	29.74	31.88	34.00	36.12	38.25	⅝	1.3	2.1
11/16	11.69	12.85	14.03	15.20	16.36	17.53	18.70	19.86	21.04	22.21	23.38	25.70	28.05	30.39	32.72	35.06	37.40	39.72	42.08	11/16	1.4	2.3
¾	12.75	14.03	15.30	16.58	17.85	19.13	20.40	21.68	22.96	24.23	25.50	28.05	30.60	33.16	35.71	38.26	40.80	43.36	45.92	¾	1.5	2.5
13/16	13.81	15.19	16.58	17.95	19.34	20.72	22.10	23.48	24.86	26.24	27.62	30.40	33.15	35.91	38.67	41.43	44.20	46.96	49.72	13/16	1.7	2.7
⅞	14.87	16.36	17.85	19.34	20.83	22.32	23.80	25.30	26.78	28.26	29.75	32.72	35.70	38.68	41.65	44.62	47.60	50.60	53.56	⅞	1.8	2.9
15/16	15.94	17.53	19.13	20.72	22.32	23.91	25.50	27.10	28.69	30.28	31.88	35.06	38.25	41.44	44.63	47.82	51.00	54.20	57.38	15/16	1.9	3.1
1	17.00	18.70	20.40	22.10	23.80	25.50	27.20	28.90	30.60	32.30	34.00	37.40	40.80	44.20	47.60	51.00	54.40	57.80	61.20	1	2.0	3.4
1 1/16	18.06	19.87	21.68	23.48	25.29	27.10	28.90	30.70	32.52	34.32	36.12	39.74	43.35	46.96	50.57	54.20	57.80	61.40	65.02	1 1/16	2.2	3.6
1⅛	19.13	21.04	22.95	24.87	26.78	28.68	30.60	32.52	34.43	36.34	38.25	42.08	45.90	49.72	53.55	57.37	61.20	65.04	68.85	1⅛	2.3	3.8
1 3/16	20.19	22.21	24.23	26.24	28.26	30.28	32.30	34.32	36.34	38.36	40.38	44.42	48.45	52.48	56.52	60.56	64.60	68.64	72.68	1 3/16	2.4	4.0
1¼	21.25	23.38	25.50	27.62	29.75	31.88	34.00	36.12	38.26	40.37	42.50	46.76	51.00	55.25	59.50	63.76	68.00	72.26	76.50	1¼	2.6	4.2
1 5/16	22.32	24.54	26.78	29.01	31.23	33.48	35.70	37.93	40.16	42.40	44.64	49.08	53.55	58.02	62.47	66.95	71.40	75.86	80.33	1 5/16	2.7	4.4
1⅜	23.38	25.71	28.05	30.39	32.72	35.06	37.40	39.74	42.08	44.41	46.75	51.42	56.10	60.77	65.45	70.12	74.80	79.48	84.15	1⅜	2.8	4.6
1 7/16	24.44	26.88	29.33	31.77	34.21	36.66	39.10	41.54	44.00	46.44	48.88	53.76	58.65	63.54	68.42	73.32	78.20	83.08	88.00	1 7/16	2.9	4.8
1½	25.50	28.05	30.60	33.15	35.70	38.26	40.80	43.35	45.90	48.45	51.00	56.10	61.20	66.30	71.40	76.51	81.60	86.70	91.80	1½	3.1	5.1
1 9/16	26.57	29.22	31.88	34.53	37.19	39.84	42.50	45.16	47.82	50.48	53.14	58.42	63.75	69.06	74.38	79.69	85.00	90.31	95.63	1 9/16	3.2	5.3
1⅝	27.63	30.39	33.15	35.91	38.67	41.44	44.20	46.96	49.73	52.49	55.25	60.78	66.30	71.83	77.35	82.88	88.40	93.93	99.45	1⅝	3.3	5.5
1 11/16	28.69	31.55	34.43	37.30	40.16	43.03	45.90	48.76	51.64	54.51	57.38	63.10	68.85	74.59	80.33	86.06	91.80	97.54	103.28	1 11/16	3.4	5.7
1¾	29.75	32.73	35.70	38.68	41.65	44.63	47.60	50.58	53.56	56.53	59.50	65.45	71.40	77.35	83.30	89.25	95.20	101.15	107.10	1¾	3.6	5.9
1 13/16	30.81	33.89	36.98	40.05	43.14	46.22	49.30	52.38	55.46	58.54	61.62	67.80	73.95	80.11	86.28	92.44	98.60	104.76	110.93	1 13/16	3.7	6.1
1⅞	31.87	35.06	38.25	41.44	44.63	47.82	51.00	54.20	57.38	60.56	63.75	70.12	76.50	82.88	89.25	95.63	102.00	108.38	114.75	1⅞	3.8	6.3
1 15/16	32.94	36.23	39.53	42.82	46.12	49.41	52.70	56.00	59.29	62.58	65.88	72.46	79.05	85.64	92.23	98.81	105.40	111.99	118.58	1 15/16	3.9	6.5
2	34.00	37.40	40.80	44.20	47.60	51.00	54.40	57.80	61.20	64.60	68.00	74.80	81.60	88.40	95.20	102.00	108.80	115.60	122.40	2	4.1	6.8

CHAPTER IX.—CAST IRON COLUMNS.

Cast iron columns are rarely used for very high skeleton construction. They should never be so used when there is great eccentricity in the loading.* They are widely used for buildings of medium height, where outer bearing walls are provided or where a sufficient distribution of substantial inclosure and curtain walls exists to brace the building adequately against lateral forces.

Cast iron columns do not lend themselves readily to the attachment of wind bracing, and because of the loose connections of the beams they should seldom be used where wind bracing is necessary.

Different forms of cross section are used for cast iron columns. The square section is used a great deal where the column is to be built into a wall; the round section is generally used for columns standing free in a room; and the other form of section the H shape is very popular for either wall or free columns—the latter only when it is to be encased in brick or plaster work. This section has the advantage of being open for inspection and it is easy to core. A special section of the same class—designed by the author—is shown in Fig. 20. It is suitable for large size columns, in fact, it is simply a practical extension of the use of the H section beyond the limits of the simple form. For small posts or struts such as stair posts and the like, the star section (Fig. 23) is sometimes used.

IN THE DESIGN OF A CAST IRON COLUMN three steps are taken: *first*, the ratio of slenderness is fixed within certain limits; *second*, the area of the section is found; and *third* the section is designed.

The *first* step has already been described in Chapter VII and the Diagrams Nos. 30 and 32 (as well as Diagram No. 34) found in that chapter will be used in conjunction with the diagrams in this chapter.

The *second* and *third* steps will be fully explained in the descriptions of Diagrams Nos. 35 and 36.

Diagram No. 35 is based on the provisions for allowable stresses contained in the "Code" (N. Y. C.). Diagram No. 36

*Small eccentricities of loading are sometimes allowed by increasing the area of the section enough to take care of the bending stress due to the eccentricity.

embodies what the author considers to be more conservative values* for use in general practice. For a column ratio of 10 the allowed stress is just the same as for the preceding diagram, but the reduction of stress with increasing ratio of slenderness is much greater than that provided by the "Code."

DIAGRAMS NOS. 35 AND 36:—The construction of these diagrams is similar to that of No. 33 for steel columns. Abscissas represent column ratios, ordinates represent areas and the loads are represented by the curves on the diagram. A supplementary ordinate scale at the right of these diagrams also gives the weight per lineal foot for any area of cross section. This value is given in the tables following these diagrams in preference to the area of the section because of its double value for both design and estimates.

An example will best illustrate the application of diagrams to the design of cast iron columns loaded with concentric and eccentric loads.

Example:—A column 10 ft. long has a load of 75 tons, 8 tons of which is located 6 in. from the neutral axis perpendicular to the web. The column section is the H as shown in Fig. 19 and is assumed to be 10 ins. square.

Solution:—The center of gravity of the combined concentric and eccentric load is 0.64 in. from the neutral axis perpendicular to the web. The coefficient of eccentricity is 5 times 0.64 or 3.2. The radius of gyration is 4 ins. about the axis perpendicular to the web, in which case the area required for this eccentricity is 20% of what would be required for the same load concentrically located, thus for the above load eccentrically located an equivalent concentric load is 120% of 75 or 90 tons.

On Diagram No. 36 the area required for a load of 9.0 tons on a column with a ratio of slenderness of 50 is 3.15 sq. ins., therefore, for a column load of 90 tons the area would be 31.5 sq. ins. This same diagram gives the weight per lineal foot of column section for the above load as 98.5 pounds.

According to Table No. 26 a 10 × 10 column 1¼ in. metal is the nearest to this requirement.

*The reader is referred to a valuable discussion on the strength of cast iron columns in "Kent's Mechanical Engineer's Pocket Book," pp. 250-252.

Diagram No. 35

For giving the safe loads on cast-iron columns as specified by the New York Building Code.

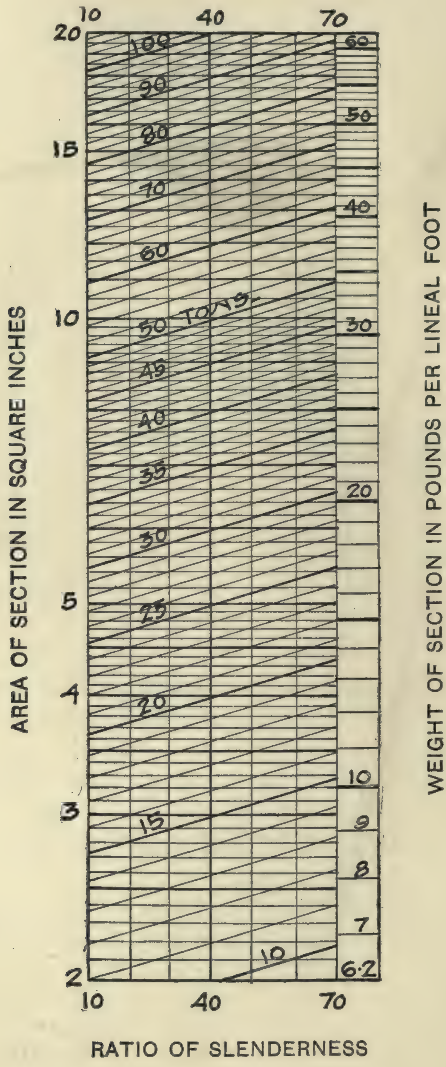
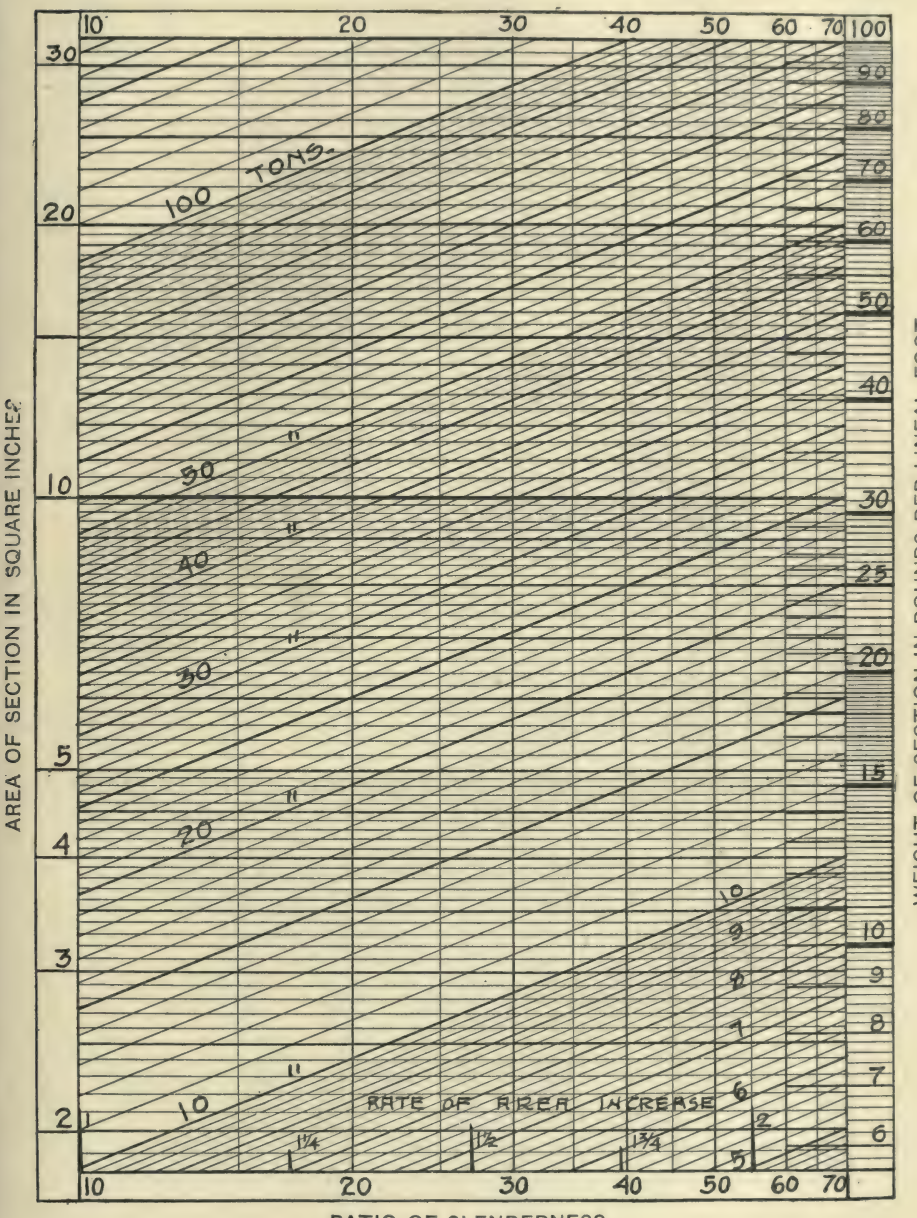


Diagram No. 36

For giving the safe loads on cast-iron columns as recommended by the author.



CAST IRON COLUMN SECTIONS

Fig. 19

WEIGHT IN POUNDS PER FOOT

Table No. 26

Size	Thickness of Metal						
	½	¾	1	1¼	1½	1¾	2
5 x 5 *	21.8	31.5	40.6	48.6			
6 x 6	26.5	38.6	49 9	60.5			
6 x 8	......	43.4	56.2	64.8			
7 x 7	......	45.5	59.3	72.0	84.2		
7 x 9	......	......	65.6	80.2	89 5		
8 x 8	......	52.7	68.6	84.0	98.2		
8 x 10	......	......	75.0	91.6	107.8		
9 x 9	......	59.8	78.0	95.5	112.0		
9 x 10	......	......	81.2	96.4	117.0		
10 x 10	......	......	87.5	108.0	126.0		
10 x 12	......	......	93.6	115 0	136.0	155.8	175.0
12 x 12	......	......	106.0	130.0	155.0	177.8	200.0

* Second dimension is in direction of web.

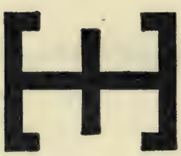


Fig. 20

WEIGHT IN POUNDS PER FOOT

Table No. 27

Size	Thickness of Metal								
	1	1¼	1½	1¾	2	2¼	2½	2¾	3
12 x 16 *	160	199	237	275	313				
20	172	214	256	297	338				
24	185	230	275	319	363				
16 x 16	197	245	293	341	388	434			
20	210	261	312	363	413	462			
24	222	276	331	385	438	490			
20 x 20	....	....	369	428	487	547	606		
24	....	....	388	450	512	575	637		
28	....	....	407	472	537	603	669	734	798
24 x 24	....	....	443	515	588	660	725	802	873
28	....	....	462	537	613	688	762	836	910
24 x 28	....	....	490	575	663	753	840	931	1023

* Second dimension is in direction of web.

CAST IRON COLUMN SECTIONS

Fig. 21

WEIGHT IN POUNDS
PER FOOT

Table 28

Outside Diameter	Thickness of Metal									
	¾	1	1¼	1½	1¾	2	2¼	2½	2¾	3
5	31.2	39.1								
6	38.6	49 0	58.2							
7	45.9	58.8	70.5							
8	53.3	68.6	82.7							
9	60.6	78.4	95.0	100.3						
10	.....	88 2	107.2	125.0	141.5					
11	......	98.0	119.5	139.7	158.7					
12	......	107.8	131.7	154.4	175.8	196.0				
13	......	......	144 0	169.0	193.0	215.5				
14	......	......	156.2	189.0	210.0	235.5				
15	......	......	168.5	198.5	227.5	255.0	281.5			
16	......	......	......	213.3	244.5	275.0	303.0	331.0		
18	......	......	......	243.0	279.0	314.0	348.0	380.0	411.0	441.0
20	......	......	......	......	313.0	253.0	392.0	429.0	465.0	500.0

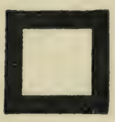


Fig. 22

WEIGHT IN POUNDS
PER FOOT

Table 29

Size	Thickness of Metal									
	¾	1	1¼	1½	1¾	2	2¼	2½	2¾	3
6 x 6	49	63	74							
8	59	75	90							
10	68	88	105							
12	77	100	121							
16	96	125	152							
8 x 8	68	88	105							
10	77	100	121							
12	87	112	137							
16	105	137	168							
10 x 10	87	112	137	159						
12	96	125	152	178						
16	115	150	183	215						
12 x 12	......	137	168	197	224	250				
16	......	162	199	234	268	300				
16 x 16	......	187	230	272	311	350	386			
20	......	......	261	309	355	400	442			
24	......	......	......	346	399	450	499	546		
20 x 20	......	......	......	346	399	450	499	546		
24	......	......	......	384	442	499	555	609		
24 x 24	......	......	......	421	486	549	611	671	730	787

CAST IRON COLUMN SECTIONS

Fig. 23

WEIGHT IN POUNDS
PER FOOT

Table 30

Size	Thickness of Metal					
	3/8	1/2	5/8	3/4	7/8	1
3″ x 3″	6.6	8.6	10.5	12.4		
4 x 4	9.0	11.7	14.4	17.0		
5 x 5		14.9	18.3	21.7	25.0	28.1
6 x 6		18.0	22.3	26.4	30.4	34.4

CAST IRON GAS PIPE

WEIGHT IN POUNDS
PER FOOT

Table 31

Size	Thickness of Metal					
	3/8	13/32	7/16	15/32	1/2	9/16
2″ φ	6.96					
3″	......	11.16				
4″	......	......	15.84			
5″	......	......	21.00			
6″	......	......	......	26.64		
8″	......	......	......	......	39.36	
10″	......	......	......	......	......	54.0

Part IV. Miscellaneous.

CHAPTER X. LOADS.

Before the framework of a building can be designed, the external forces which act on the structure, i. e., the loads, must be known or assumed. In this chapter are given some average values which may be used for the different items in estimating loads, together with the principal provisions regarding loading which are found in the "Code" (N. Y. C.).

Loads are usually divided into Dead Loads, Live Loads and Wind Loads. Sometimes wind loads are included under live loads.

DEAD LOADS.

THE DEAD LOADS CARRIED BY FLOOR GIRDERS consist of: the steel beams, connection angles, tie rods and other fittings; floor arches, nailing strips, filling and finished flooring; partitions, stairs and other permanent construction; and suspended ceilings, pipes and conduits.

THE DEAD LOADS CARRIED BY COLUMNS consist of: the dead loads from the floor or floors supported by the column; the column itself, including the metal and covering material; all pipes and conduits suspended to the column; and loads from trusses or girders carrying brick walls, vaults or other permanent loads.

THE WEIGHT OF STEEL FLOOR FRAMING may be determined approximately from the two accompanying Diagrams Nos. 37 and 38. Diagram No. 37 applies to floors intended to carry a total load (live and dead) of about 150 lbs. per sq. ft. Diagram No. 38 applies to floors intended to carry a total load of about 300 lbs. per sq. ft. In these diagrams abscissas represent span in feet, and ordinates represent weight of steel per square foot of floor. Two sets of lines are drawn on each diagram: the upper set consisting of three lines is for floor beams spaced 3, 4 or 5 ft.; the lower five lines are for floor girders, representing spacings of 10, 15, 20, 25 and 30 ft.

To use either of the two diagrams, take an abscissa equal to

Diagram No. 37

For giving the weight of steel required in floors where the loads are a minimum.

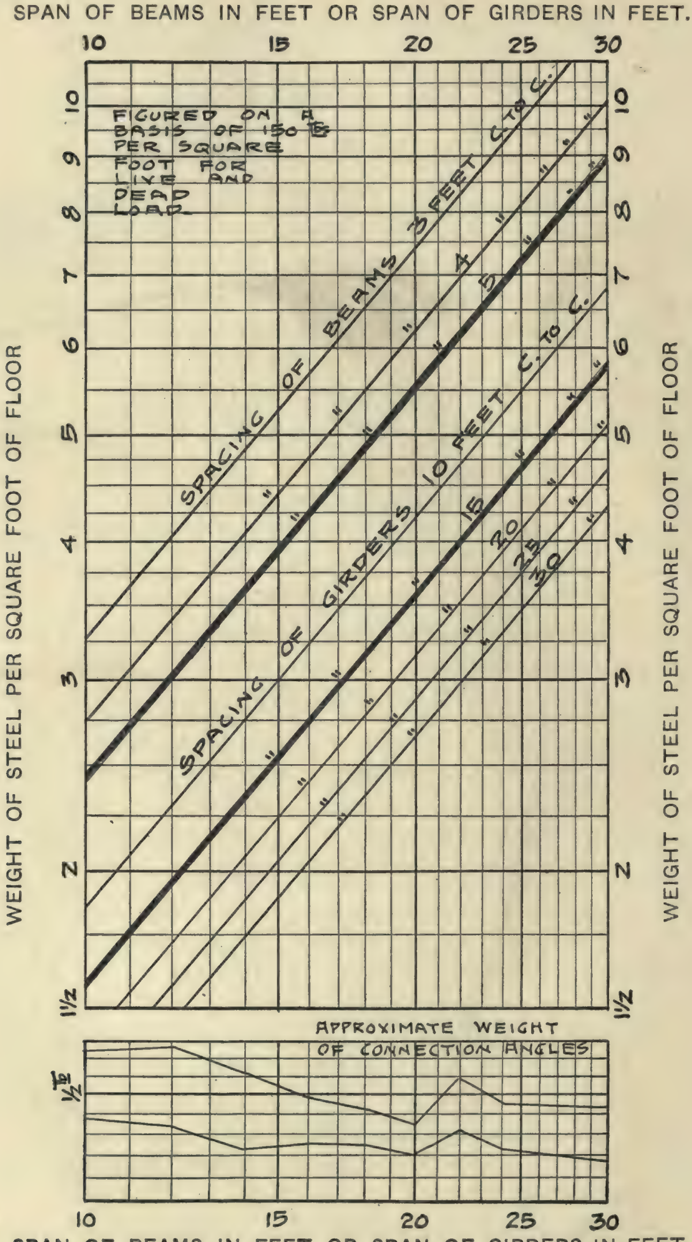


Diagram No. 38

For giving the weight of steel required in floors where the loads are a maximum.

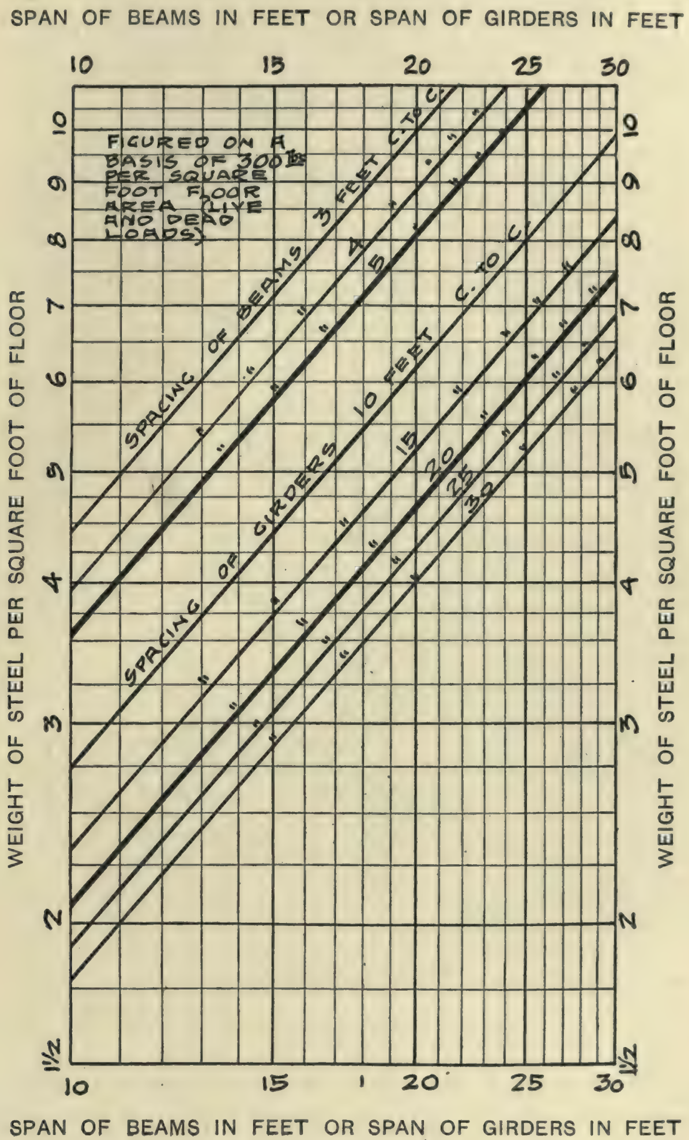
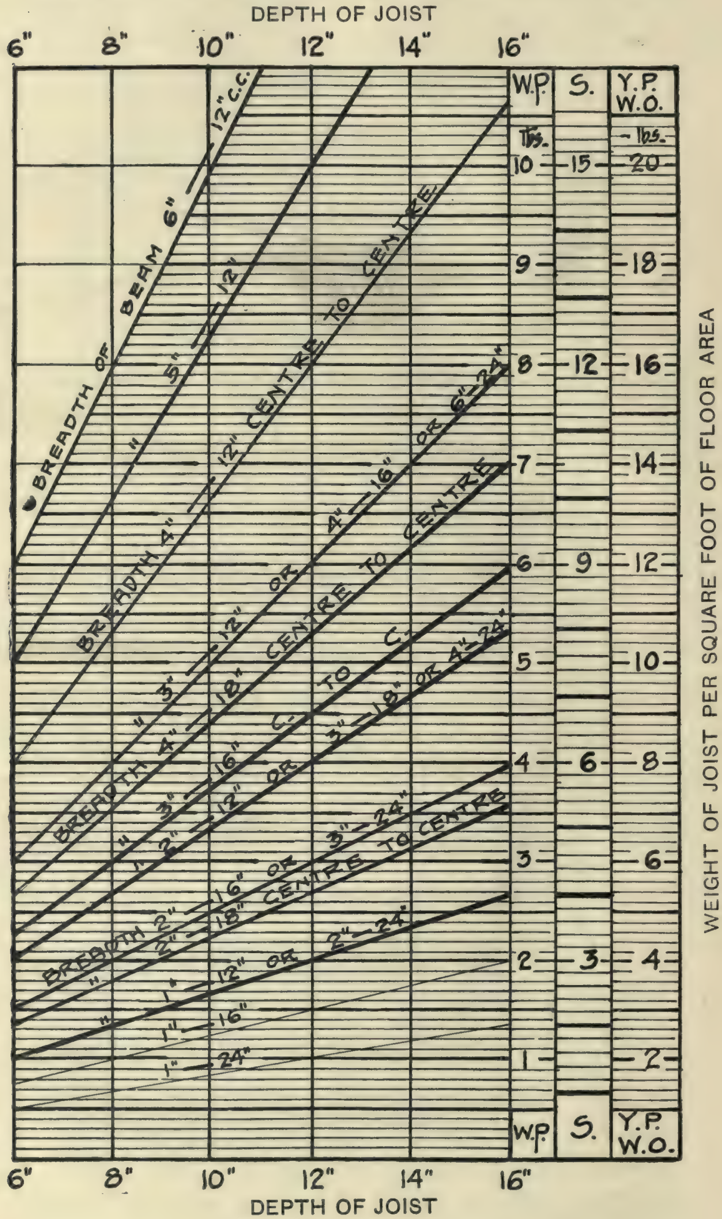


Diagram No. 39

For giving the weight of joists per square foot of floor.



the span of the floor beams in feet and follow up to the diagonal representing the assumed spacing of the beams in feet. The horizontal at the intersection gives the weight of floor beams in pounds per square foot of floor. Then take an abscissa equal to the span of the girders and follow to the diagonal representing the spacing —the horizontal at the intersection gives the weight of the girders per square foot of floor. The sum of the weights of the beams and the girders gives the total weight of steel per square foot of floor.

At the bottom of Diagram No. 37 are two curves which represent approximately the limiting weights of connection angles per square foot of floor for different spans of beam. Evidently these weights are too small to affect the loading.

THE WEIGHT OF WOODEN FLOOR BEAMS may be determined approximately from Diagram No. 39 herewith. In this diagram abscissas represent depth of the beam and ordinates represent weight per square foot of floor. The diagonal lines on the diagram represent different breadths of beam, and different spacing of beams. The ordinates may be measured on one of three scales at the right of the diagram: these scales give the weights per square foot of floor respectively for white pine, spruce and yellow pine or white oak.

FLOOR ARCHES:—The weight of tile arches varies from 18 to 40 lbs. per sq. ft. according to the depth, density of tile, thickness, and distance of webs apart. About 5 lbs. per sq. ft. should be added to the weight of floor arches for the mortar in the joints. The weight of concrete arches varies from 18 to 40 lbs. per sq. ft. according to the span of the arch and to the system of construction* adopted.

FLOORING MATERIAL:—Hardwood floors weigh about 4 lbs. per sq. ft., for every inch in thickness, and softwood about 3 lbs. Nailing strips weigh about 2 lbs. per sq. ft. of floor. Tile or terrazza floors weigh about 14 lbs. per sq. ft. for every inch in thickness.

Concrete filling weighs from 6 to 8 lbs. per sq. ft. for every inch in thickness according to composition.

Plastered ceilings weigh about 8 lbs. per sq. ft. In case of suspended ceilings, the weight of the steel work and tile or lath must be added.

*Valuable information on the various flooring systems will be found in Frietag's "Architectural Engineering" and Kidder's "Building Construction and Superintendence," Part I.

PARTITIONS:—Tile plastered on both sides weighs from 24 to 40 lbs. per sq. ft. of wall surface. Expanded metal weighs from 18 to 22 lbs. per sq. ft.

BRICK WALLS:—The weight of walls will be found on Diagram No. 40 in Chapter XII.

LIVE LOADS.

Live or variable loads consist of all loads other than dead loads. The "Code" (N. Y. C.) provides that:—

"Every floor shall be of sufficient strength to bear safely the weight to be imposed thereon in addition to the weight of the materials of which the floor is composed. Thus:—

Floor Loads per sq. ft.	
Dwelling house	60 lbs.
Apartment house	60 "
Tenement house	60 "
Hotel or lodging house	60 "
Office buildings:	
Above 1st floor	75 "
1st floor	150 "
School or place of instruction	75 "
Stable or carriage house	75 "
Place of public assembly	90 "
Ordinary stores, light manufacturing and light storage	120 "
Store where heavy materials are kept, warehouse or factory	150 "
Roofs pitch less than 20°	50 "
pitch more than 20°	30 "
(load on a horizontal plane.)	
Sidewalks between the curb and area lines	300 "

"For the purpose of determining the carrying capacity of columns in dwellings, office buildings, stores, stables and public buildings when over five stories in height, a reduction of the live loads shall be permissible as follows:

"For the roof and top floor the full live loads shall be used; for each succeeding lower floor it shall be permissible to reduce the live load by 5% until 50% of the live loads fixed by this section is reached, when such reduced loads shall be used for all remaining floors."

The strength of factory floors intended to carry running machinery should be increased about 50% above the preceding provisions.

Regarding the reduction of live loads on columns, as specified in the above quotation, it may be remarked that this does not

take into account the *area* of floor tributary to the column as affecting the allowable percentage of reduction—a rational provision should allow for this.

LIVE LOADS ON FOOTINGS:—According to the "Code" (N. Y. C.) the loads exerting pressure under the footings of foundations in buildings more than three stories in height are to be computed as follows:

"For warehouses and factories they are to be the full dead and the full live load. In stores and buildings for light manufacturing purposes they are to be the full dead load and 75% of the live load. In churches, school houses and places of public amusement, they are to be the full dead load and 75% of the live load. In office buildings, hotels, dwellings, apartment houses, tenement houses, lodging houses and stables, they are to be the full dead load and 60% of the live load. Footings shall be so designed that the loads will be as nearly uniform as possible and not in excess of the safe bearing capacity of the soil."

WIND PRESSURE ON BUILDINGS:—The wind pressure allowed for on buildings is very much a matter of guess work. The provisions of the "Code" (N. Y. C.) however, are recognized as representing safe limits. These provide that:

"All structures exposed to wind shall be designed to resist a horizontal wind pressure of 30 lbs. for every sq. ft. of surface thus exposed, from the ground to the top of same including roof, in any direction. In no case shall the over-turning moment due to wind pressure exceed 75% of the moment of stability of the structure. In all structures exposed to wind, if the resisting moments of the ordinary materials of construction, such as masonry, partitions, floors and connections, are not sufficient to resist the moment of distortion due to wind pressure, taken in any direction on any part of the structure, additional bracing shall be introduced sufficient to make up the difference in moments. In calculations for wind bracing, the working stresses set forth in this code may be increased by 50%. In buildings under 100 ft. in height, provided the height does not exceed four times the average width of the base, the wind pressure may be disregarded."

CHAPTER XI. UNIT STRESSES.

The allowable working stresses to be used in designing are of fundamental importance. In the construction of the diagrams and tables throughout this book the following stresses were considered. As previously stated these stresses were taken from the "Code" (N. Y. C.) and because of their importance they are collected together in this Chapter for general use and reference.

SAFE LOAD ON MASONRY WORK.—The safe bearing load in tons per superficial foot shall be taken at

8 for brickwork in lime mortar,
11½ for brickwork lime and cement mixed,
15 for brickwork cement mortar
10 " rubble-stone work in Portland cement mortar
8 " " " cement mortar
7 " " " lime and cement
5 " " " lime mortar
15 " Portland cement concrete
8 " cement concrete (natural)

STRENGTH OF COLUMNS:—In columns or compression members with flat ends of cast iron, steel, wrought iron or wood, the stress per square inch shall not exceed that given in the following tables:

When the length divided by least radius of gyration equals	Working stresses per square inch of section.		
	Cast iron.	Steel.	Wrought iron.
120	......	8,240	4,400
110	......	8,820	5,200
100	......	9,400	6,000
90	......	9,980	6,800
80	......	10,560	7,600
70	9,200	11,140	8,400
60	9,500	11,720	9,200
50	9,800	12,300	10,000
40	10,100	12,880	10,800
30	10,400	13,460	11,600
20	10,700	14,040	12,400
10	11,000	14,620	13,200

When the length divided by the least diameter equals	Working stresses per square inch of section.		
	Long leaf yellow pine.	White pine, Norway pine, Spruce	Oak.
30	460	350	390
25	550	425	475
20	640	500	560
15	730	575	645
12	784	620	696
10	820	650	730

And in like proportion for intermediate ratios. Five-eighths the values given for white pine shall apply to chestnut and hemlock posts. For locust posts use 1½ the value given for white pine.

Columns and compression members shall not be used having an unsupported length of greater ratios than given in the tables.

WORKING STRESSES:—The safe carrying capacity of the various materials of construction (except in the case of columns) shall be determined by the following working stresses in pounds per square inch of section area:

COMPRESSION (DIRECT).

			With grain.	Across grain.*
Rolled steel	16,000			
Cast steel	16,000	Oak	900	500
Wrought iron	12,000	Yellow pine	1,000	350
Cast iron (in short blocks)	16,000	White pine	800	200
Steel pins and rivets (bearing)	20,000	Spruce	800	200
		Locust	1,200	...
Wrought iron pins and rivets (bearing)	15,000	Hemlock	500	150
		Chestnut	500	250

Concrete (Portland) cement, 1; sand, 2; stone, 4	230
Concrete (Portland) cement, 1; sand, 2; stone, 5	208
Concrete, Rosendale, or equal, cement, 1; sand, 2; stone, 4	125
Concrete, Rosendale, or equal, cement, 1; sand, 2; stone, 5	111
Rubble stonework in Portland cement mortar	140
Rubble stonework in Rosendale cement mortar	111
Rubble stonework in lime and cement mortar	97
Rubble stonework in lime mortar	70
Brickwork in Portland cement mortar; cement, 1; sand, 3	250
Brickwork in Rosendale, or equal, cement mortar; cement, 1; sand, 3	208
Brickwork in lime and cement mortar; cement, 1; lime, 1; sand, 6	160
Brickwork in lime mortar; lime, 1; sand, 4	111
Granites (according to test)	1,000 to 2,400
Greenwich stone	1,200
Gneiss (New York City)	1,300
Limestones (according to test)	700 to 2,300
Marbles (according to test)	600 to 1,200
Sandstones (according to test)	400 to 1,600
Bluestone, North River	2,000
Brick (Haverstraw, flatwise)	300
Slate	1,000

*These values for compression across the grain have been revised by the author.

TENSION (DIRECT).

Rolled steel	16,000	White pine	800
Cast steel	16,000	Spruce	800
Wrought iron	12,000	Oak	1,000
Cast iron	3,000	Hemlock	600
Yellow pine	1,200		

SHEAR.

Steel web plates	9,000
Steel shop rivets and pins	10,000
Steel field rivets	8,000
Steel field bolts	7,000
Wrought iron web plates	6,000
Wrought iron shop rivets and pins	7,500
Wrought iron field rivets	6,000
Wrought iron field bolts	5,500
Cast iron	3,000

	With fiber.	Across fiber.
Yellow pine	70	500
White pine	40	250
Spruce	50	320
Oak	100	600
Locust	100	720
Hemlock	40	275
Chestnut	..	150

SAFE EXTREME FIBER STRESS (BENDING).

Rolled steel beams	16,000
Rolled steel pins, rivets and bolts	20,000
Riveted steel beams (net flange section)	14,000
Rolled wrought-iron beams.	12,000
Rolled wrought-iron pins, rivets and bolts	15,000
Riveted wrought-iron beams (net flange section)	12,000
Cast-iron, compression side.	16,000
Cast-iron, tension side	3,000
Yellow pine	1,200
White pine	800
Spruce	800
Oak	1,000
Locust	1,200
Hemlock	600
Chestnut	800

Granite	180
Greenwich stone	150
Gneiss (New York City)	150
Limestone	150
Slate	400
Marble	120
Sandstone	100
Blue stone, North River	300
Concrete (Portland) cement, 1; sand, 2; stone, 4	30
Concrete (Portland) cement, 1; sand, 2; stone, 5	20
Concrete (Rosendale, or equal) cement, 1; sand, 2; stone, 4	16
Concrete (Rosendale, or equal) cement, 1; sand, 2; stone, 5	10
Brick (common)	50
Brickwork (in cement)	30

BEARING CAPACITY OF SOIL.—Where no test of the sustaining power of the soil is made, different soils, excluding mud, at the bottom of footings shall be deemed to safely sustain the following loads in tons per superficial foot, namely:

Soft clay	1.
Ordinary clay and sand together, in layers, wet and springy	2.
Loam, clay or fine sand, firm and dry	3.
Very firm, coarse sand, stiff gravel or hard clay	4.

"When a doubt arises as to the safe sustaining power of earth upon which a building is to be erected, the Department of Buildings may order borings to be made, or direct the sustaining power of the soil to be tested by and at the expense of the owner of the proposed building."

CHAPTER XII. BRICK WALLS.

The walls of a building are an important part of the loads in the case of skeleton construction. In the case of building where the walls are the principal supporting element the strength, thickness, etc., of the walls are essential items in their design. In many cities the building laws prescribe the thickness for the various conditions in the construction of walls. An especially full set of provisions of this nature is contained in the "Code" (N. Y. C.) and they are taken as a basis for this chapter. These provisions divide the walls of buildings into three classes: walls for dwelling houses, walls for warehouses and inclosure walls for skeleton structures.

WALLS FOR DWELLING HOUSES.—This class includes the following buildings:

Apartment Houses,
Asylums,
Club Houses,
Convents,
Dormitories,
Dwellings,
Hospitals,
Hotels,
Laboratories,
Lodging Houses,
Parish Buildings,
Schools,
Studios,
Tenements.

WALLS FOR WAREHOUSES.—This class includes the following buildings:

Armories,
Breweries,
Churches,
Cooperage Shops,
Court Houses,
Factories,
Foundries,
Jails,
Libraries,
Light and Power Houses,
Machine Shops,
Markets,
Mills,
Museums,
Observatories,
Office Buildings,
Police Stations,
Printing Houses,
Public Assembly Buildings,
Pumping Stations,
Railroad Buildings,
Slaughter Houses,
Stables,
Stores,
Theatres,
Warehouses,
Wheelwright Shops.

INCLOSURE WALLS FOR SKELETON STRUCTURES.—These are walls of brick built in between iron or steel columns, and supported wholly or in part on iron or steel girders.

THICKNESS AND WEIGHT OF WALLS.

The provisions of the "Code" for thicknesses of these three classes of walls have been embodied in a single diagram.

DIAGRAM NO. 40 (GIVING THICKNESS AND WEIGHT OF WALLS):—In each of the foregoing classes of walls the thickness

varies with the height above the curb level. These variations are represented in five sections on this diagram. In each section the full unshaded outline represents "warehouse walls," the dotted line represents "dwelling house walls, and the full shaded portion represents "inclosure walls for skeleton structures." The thicknesses for each case are indicated by vertical lines, each space of which represents a thickness equal to one half the length of a brick or about 4 ins. On three of these sections is given the weight of wall per lineal foot for the different vertical heights at which the figures for the weights are placed. These weights apply to "warehouse walls." A supplementary scale at the right of the diagram gives the weight per lineal foot of skeleton inclosure walls for different heights below the top. A farther ordinate scale, at the extreme right, gives the number of bricks in such skeleton walls, per lineal foot of wall.

Three supplementary scales at the left of the diagram will be found useful. They show:—

(1) Number of stories, at different levels, if stories are 10 ft. from floor to floor.

(2) The same if stories are 12½ ft. from floor to floor.

(3) The floor area gained, per lineal foot of wall, by using skeleton walls instead of warehouse walls; this latter is for the accumulated gain for all the floors—the floors are assumed to be 12½ ft. apart vertically.

CONDITIONS THAT WILL MODIFY THE FOREGOING PROVISIONS.—"If there is a clear span of over 25 ft. between the bearing walls* of warehouses (26 ft. for dwelling houses), such walls shall be 4 ins. more in thickness than in this section specified, for every 12½ ft. or fraction thereof, that said walls are more than 25 ft. apart (26 ft. for dwellings), or shall have, instead of the increased thickness, piers or buttresses."

"All buildings, not excepting dwellings, that are over 105 ft. in depth, without a cross wall or proper piers or buttresses, shall have the side or bearing walls increased in thickness 4 ins. more than is specified in the respective sections of this code for the thickness of walls, for every 105 ft., or part thereof, that the said buildings are over 105 ft. in depth."

"If any horizontal section through any part of any bearing wall in any building shows more than 30% area of flues and openings, the said wall shall be increased 4 ins. in thickness for every 15% or fraction thereof, of flue or opening area in excess of 30%."

"Non-bearing walls may be 4 ins. less in thickness, provided none are less than 12 ins., for warehouses and dwellings."

*Bearing walls are those walls on which beams, girders or trusses rest.

FOUNDATION WALLS.

"When either bearing or skeleton walls are supported, the foundation walls, if built of rubble-stone or Portland cement concrete, shall be at least 8 ins. thicker than the wall next above them, to a depth of 12 ft. below the curb level; and for every additional 10 ft., or part thereof, deeper, they shall be increased 4 ins. in thickness. If built of brick, they shall be at least 4 ins. thicker than the wall next above them to a depth of 12 ft. below the curb level; and for every additional 10 ft. or part thereof, deeper, they shall be increased 4 ins. in thickness. The footing or base course shall be of stone or of concrete, or both, or of concrete and stepped-up brickwork, of sufficient thickness and area to safely bear the weight to be imposed thereon."

"The thickness of a RETAINING WALL at its base shall be in no case less than one-fourth of its height."

BRICK WALLS FOR SKELETON CONSTRUCTION.

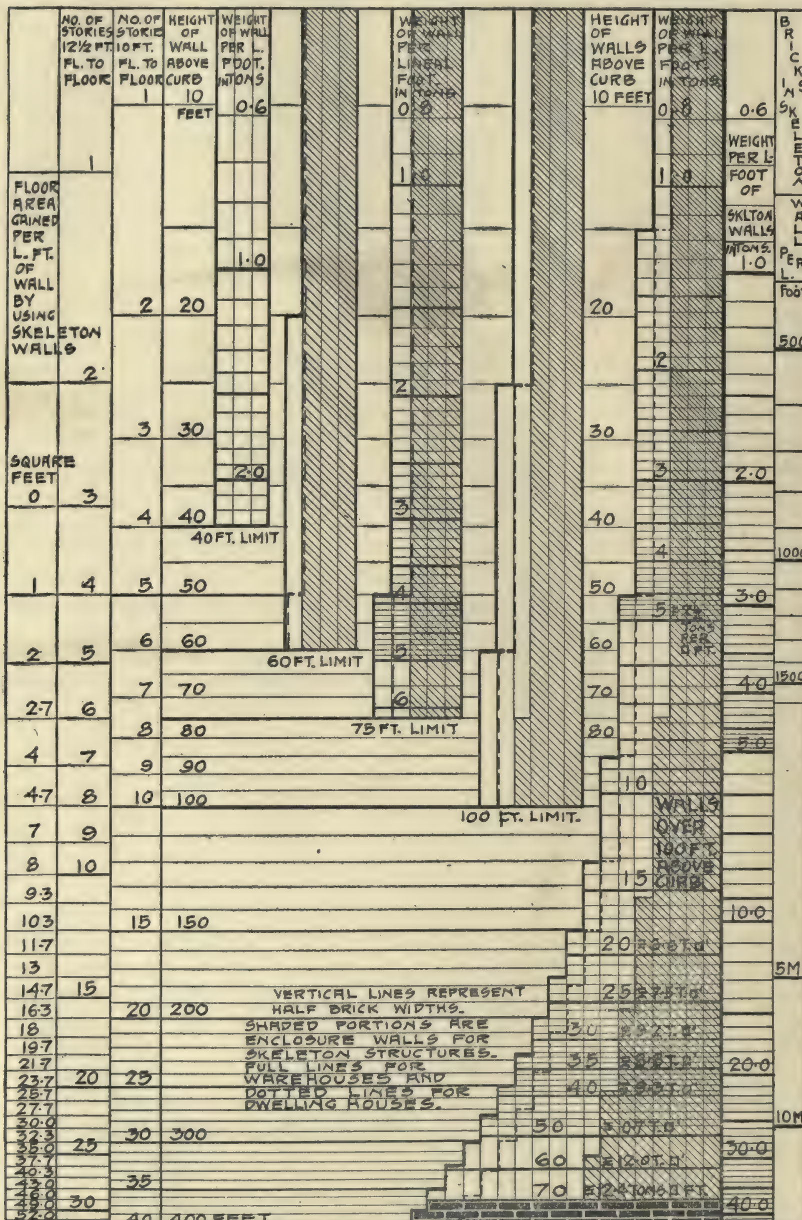
"Where columns are used to support iron or steel girders carrying inclosure walls, the said columns shall on their exposed outer and inner surfaces be constructed to resist fire by having a casing of brickwork not less than 8 ins. in thickness on the outer surfaces, nor less than 4 ins. on the inner surfaces, and all bonded into the brickwork of the inclosure walls.

"The exposed sides of the iron or steel girders shall be similarly covered in with brickwork not less than 4 ins. in thickness on the outer surfaces and tied and bonded, but the extreme outer edge of the flanges of beams, or plates, or angles connected to the beams, may project to within 2 ins. of the outside surfaces of the brick casing.

"The inside surface of girders may be similarly covered with brickwork, or if projecting inside of the wall, they shall be protected by terra cotta, concrete or other fireproof material."

Diagram No. 40.

For giving the thickness and weight of walls for skeleton structures, warehouses or dwelling houses according to the New York Building Code.



CHAPTER XIII. GERMAN, BELGIAN AND ENGLISH I-BEAMS.

It may be of interest to the designer to compare foreign practice in rolled beams with American practice. For this reason Tables Nos. 32, 33 and 34, have been prepared, giving the principal dimensions of I-beams made in Germany, Belgium and England respectively.

GERMAN I-BEAMS:—This list of shapes is a most interesting series from a theoretical point of view. The increments of increase of weight, thickness of web and strength follow almost a perfect law of symmetry. From a practical aspect there is little of value in them that would appeal to the American mind; there are 27 different heights between $5\frac{1}{2}$ ins. and $21\frac{5}{8}$ ins., inclusive, while the ten different heights adopted by the American Association of Steel Manufacturers in 1896 for beams 6 ins. and over already seem too many.

In this table the section-moments are given on a basis of 8 tons per sq. in. allowable fiber stress. These are given more for comparative than for practical value. In fact, if this table is used for designing beams, it would be advisable to use only 70% of the section-moment given, unless the material passes the American standard inspection.

BELGIAN I-BEAMS:—Representative Belgian I-beams are listed in Table No. 33. These shapes are somewhat nearer the sections adopted in this country; there being from two to four different beams rolled for each height. Otherwise, what has been said as to strength and adaptability of German shapes to American needs, applies to the beams of this group.

ENGLISH I-BEAMS:—Table No. 34 shows a series of English sections of I-beam. As might be expected, these shapes conform much more to the American system of rolling I-beams than any other. The values for the section moments are given on a basis of 8 tons per sq. in. allowable fiber stress.

PROPERTIES OF FOREIGN I-BEAMS

GERMAN				TABLE 32	BELGIAN				TABLE 33	ENGLISH				TABLE 34
Height of Beam	Weight per Foot	Flange Width	Web Thickness	Section-Moment	Height of Beam	Weight per Foot	Flange Width	Web Thickness	Section-Moment	Height of Beam	Weight per Foot	Flange Width	Web Thickness	Section-Moment
Inches	Lbs.	Inches	Inches	Ft.-Tons	Inches	Lbs.	Inches	Inches	Ft.-Tons	Inches	Lbs.	Inches	Inches	Ft.-Tons
5.510	9.54	2.598	.2244	3.33	1.97	2.3	0.98	0.16	0.24	3	4	1.25	0.218	0.65
5.905	10.7	2.756	.2362	3.98		3.4	1.14	.31	.31		10	3	.290	1.86
6.300	12.0	2.913	.2480	4.725	3.15	3.9	1.5	.16	.74	3.5	6	1.5	.296	1.10
6.693	13.2	3.071	.2598	5.57		6.4	1.73	.39	1.00		10.5	3	.350	2.23
7.087	14.6	3.228	.2716	6.52	3.94	6.0	1.97	.19	1.58	4	5	1.75	.180	1.21
7.480	16.0	3.386	.2834	7.51		9.25	2.20	.43	1.98		8	1.75	.331	1.72
7.874	17.5	3.543	.2952	8.70	4.72	7.5	2.28	.197	2.29		9.5	3	.225	2.51
8.268	19.0	3.701	.3071	9.9		11.25	2.52	.433	2.88		12	3	.299	3.00
8.662	20.7	3.858	.3189	11.3	5.0	11.5	2.95	.236	3.810	4.625	14	3	.400	3.78
9.055	22.4	4.016	.3307	12.72		15.5	3.19	.472	4.46	4.75	6.5	1.75	.187	1.82
9.450	24.1	4.173	.3425	14.35		18.75	4.55	.314	6.51		10	1.75	.400	2.38
9.842	26.0	4.331	.3544	16.10		22.9	4.76	.550	7.16	5	11	3	.230	3.64
10.236	28.0	4.448	.3701	17.91	5.5	9.6	2.60	.224	3.73		15	3	.400	4.28
10.630	29.9	4.567	.3819	19.91		14.1	2.83	.460	4.53		19	4.18	.440	5.88
11.022	32.0	4.685	.3976	22.75		11.5	2.76	.275	4.04		22	4.5	.342	7.18
11.417	34.0	4.803	.4094	24.15		15.9	2.99	.511	5.42		24	5	.371	7.88
11.81	36.2	4.921	.4252	26.50	6.0	23.0	5.0	.354	9.33	5.25	9	1.5	.368	2.22
12.6	40.7	5.158	.4527	31.75		27.75	5.25	.590	10.26	5.5	10.5	2	.329	3.12
13.385	45.4	5.394	.4803	37.41	6.3	12.0	2.91	.248	4.62	6	12	2	.381	3.68
14.17	50.9	5.630	.5118	44.15		17.1	3.13	.484	5.65		13	3	.322	4.61
14.96	56.0	5.866	.5394	51.30		15.5	3.13	.315	6.20		16	3	.390	5.58
15.75	61.7	6.102	.5670	59.30		20.5	3.38	.550	7.23		20	4.5	.434	7.40

GERMAN—Continued					BELGIAN—Continued					ENGLISH—Continued				
Height of Beam	Weight per Foot	Flange Width	Web Thickness	Section-Moment	Height of Beam	Weight per Foot	Flange Width	Web Thickness	Section-Moment	Height of Beam	Weight per Foot	Flange Width	Web Thickness	Section-Moment
Inches	Lbs.	Inches	Inches	Ft.-Tons	Inches	Lbs.	Inches	Inches	Ft.-Tons	Inches	Lbs.	Inches	Inches	Ft.-Tons
16.73	69.2	6.418	.6024	70.60	6.9	14.1	3.13	.334	5.41	6	25	5	.423	9.65
17 715	77.3	6.693	.6378	82.80		19.6	3.38	.570	6.66	6.25	18	3.5	.339	7.05
18.70	85 3	7.008	.6732	96.40	7.08	17.5	3.54	.394	7.48	7	16	3.75	.250	6.85
19 685	94 1	7.284	.7086	111.50		23.0	3.78	.630	8.80		18	3.75	.313	8.0
21.652	112.0	7.974	.7480	146.10		20.25	3.94	.394	9.36	8	19	4	.329	9.4
						25.9	4.17	.630	10.70		25	4	.410	12.2
					7.87	18.25	3.54	.354	7.85		30	5	.400	15.6
						24.5	3.78	.59	9.47		35	6	.440	18.6
						20.9	3.94	.393	9.90	9	20	3.75	.300	11.1
						27.25	4.17	.63	11.52		58	7	.777	32.1
					8.0	27.0	5.00	.394	15.0	9.25	21.5	3.75	.360	11.7
						33.25	5.23	.63	16.7	9.81	36	4.5	.516	21.1
					9.25	22.25	3.54	.394	12.0	10	30	4.5	.387	18.8
						29.50	3.78	.63	14.26		29	5	.350	18.9
					9.84	29.0	4.53	.433	17.02		35	5	.480	21.3
						37.0	4.76	.670	19.55		45	6	.489	28.8
					11.81	39.5	4.92	.433	28.8	12	39	5	.440	28.3
						49.0	5.16	.669	32.4		32	5	.350	24.5
					12.01	51	5.92	.591	40		44	6	.410	34.9
						61	6.16	.827	43.8		54	6	.510	41.1
					12.48	57	6.18	.591	45.9	13	41.5	5	.510	30.8
						67	6.42	.827	50.1	14	46	6	.435	40.2
					13.97	53	5.98	.512	46.1		57	6	.510	49.6
						64.5	6.22	.748	51.2	15	42	5	.422	36.8
					15.98	56.5	5.98	.551	55.9		59	6	.540	54.8
						70.0	6.22	.797	62.6	16	50	5	.510	46.3
											62	6	.560	60.2
										18	75	7	.550	85.2
										20	89	7.5	.600	109.7

CHAPTER XIV. FLEXURAL EFFICIENCY OF I-BEAMS AND CHANNELS.

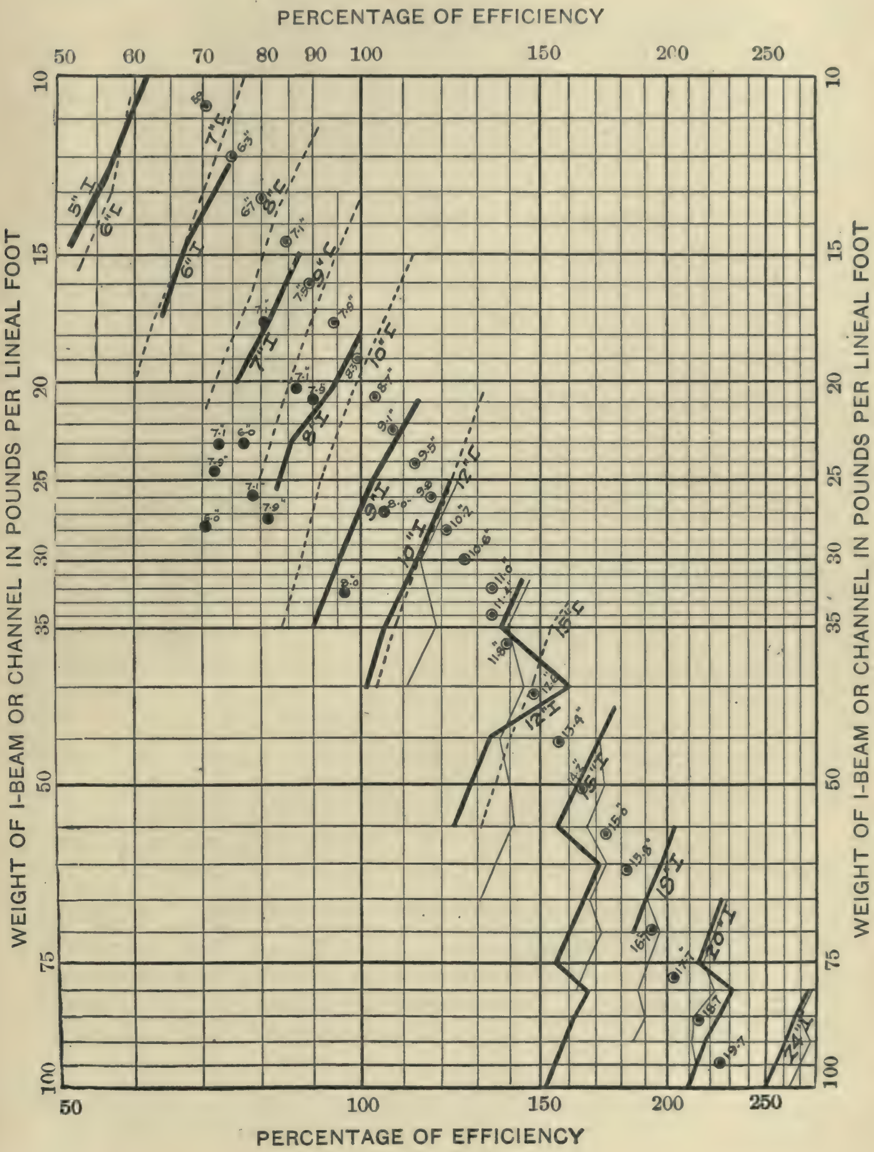
An interesting matter in connection with the design of beams and girders is the relative "efficiency" of the different standard sections. An 8-in. 18 lbs. I-beam has a certain section moment. If this beam be taken as a standard of comparison, then the value of its section-moment divided by 18 may be taken as a unit of efficiency, i. e., 100%. Thus a 20-in. 65 lb. I-beam, for instance, shows a much greater bending strength per pound of metal contained in a lineal foot of the beam.

DIAGRAM NO. 41 herewith gives curves representing the relative efficiency of all the standard and special section of I-beams and channels. In this diagram abscissas represent efficiencies, based on the above standard. Ordinates represent weights per lineal foot. Different lines drawn on the diagram represent the various sections of beams. The full heavy lines represent the I-beams of the American Association of Steel Manufacturers. The light full lines represent Pencoyd I-beams (only shown where they differ considerably from the Association standards). The dotted lines represent standard and special channels.

As a matter of comparison some foreign sections of I-beams are also represented on this diagram. The solid dots represent some of the oddest of the Belgian I-beams. The depth of the beam is in each case noted next the dot in figures representing inches. In like manner, a number of representative German sections of I-beams are shown by circles with black centers. The sizes they represent are given as for the Belgian shapes.

Diagram No. 41

For giving the relative flexure efficiency of I-beams and channels per pound of steel.



CHAPTER XV. BASES AND LINTELS OF CAST IRON.

CAST IRON BASES OR SHOES FOR COLUMNS:—These usually bear upon concrete, dimension stone, brickwork or upon grillage footings. They are usually set in place upon small blocks and grouted with Portland cement paste from one-half to three-fourths of an inch in thickness. A better practice is to ram the Portland cement mortar in from the side of the shoe, to do which requires not less than an inch and a half to two inches of clear space under the shoe when temporarily supported by the blocks.

The *area* of the bottom side of this shoe is determined by the allowable unit pressure on the supporting material. The *height* and *thickness of metal* depend upon a variety of certain and uncertain conditions. Only a few of the more important of these conditions will be considered because they arise in specific cases rather than in general practice.

A slight unevenness in the grouting of a cast iron shoe is not an unusual occurrence, and this is sure to set up irregular and indeterminate stresses in the metal. When grillage beams are used in footings the slightest deflection of the beams will cause the load on the shoe to be carried on its two edges at right angles to the beams. The distance of the webs or ribs apart should never exceed the limits fixed by the strength of the bottom plate between these webs.

A "rule of thumb" method of designing cast-iron shoes is to let

$$h = 1\tfrac{3}{4}\ a \qquad (25)$$

where h = height of shoe,

a = projection of shoe beyond the edge of the column.

The thickness of metal is made the same as that of the column supported.

The purely theoretical methods for computing the flexure strength of shoes are laborious and tedious. The following simple empirical formulas give results within a very small percentage of absolute accuracy. Two cases are considered:

(1) When a uniform unit pressure is assured on the bottom of the shoe—for instance, a bearing on a granite block.

(2) When the load is likely to be carried on two edges of the shoe—a condition existing when grillage footings are used.

FIRST CASE:—

$$A = \frac{a^2}{(2a + b)} \frac{W}{h} \qquad (26)$$

where a = projection in inches.
b = diameter of column in inches.
h = height of shoe in inches.
W = total load in tons.
A = area* of cross section (in square inches).

SECOND CASE:—

$$A = \frac{a}{h} W \qquad (27)$$

where the values are same as above.

When the thickness of metal in the various parts of a cast iron shoe is not uniform, or nearly so, flaws are apt to develop in the process of the cooling off of the casting. For this reason the design of an economical shoe requires skill in choosing the height and distance apart of the webs so that the metal in the several parts shall be one thickness. The thickness of metal directly under the column section must always be sufficient for the load on the column.

The safe limits for the distance between the webs or ribs of cast iron shoes are given on **DIAGRAM NO. 42** herewith. It gives the safe distance in inches for various unit loads on the bottom of the shoe and for various thicknesses of metal in the bottom plate.

EXAMPLE:—For a unit pressure on the bottom of a shoe of 11½ tons per sq. ft., the brackets should not be more than 10 ins. apart for 1⅝-in. metal in the bottom plate, while for 18 tons the metal should be 2 ins. for the same spacing of brackets.

CAST IRON LINTELS:—When the height of the stem of a cast iron lintel is not less than about $^4/_{10}$ths of the width of the lintel per stem, the formulas following can safely be used. Two stems should be used for widths greater than 16 ins.

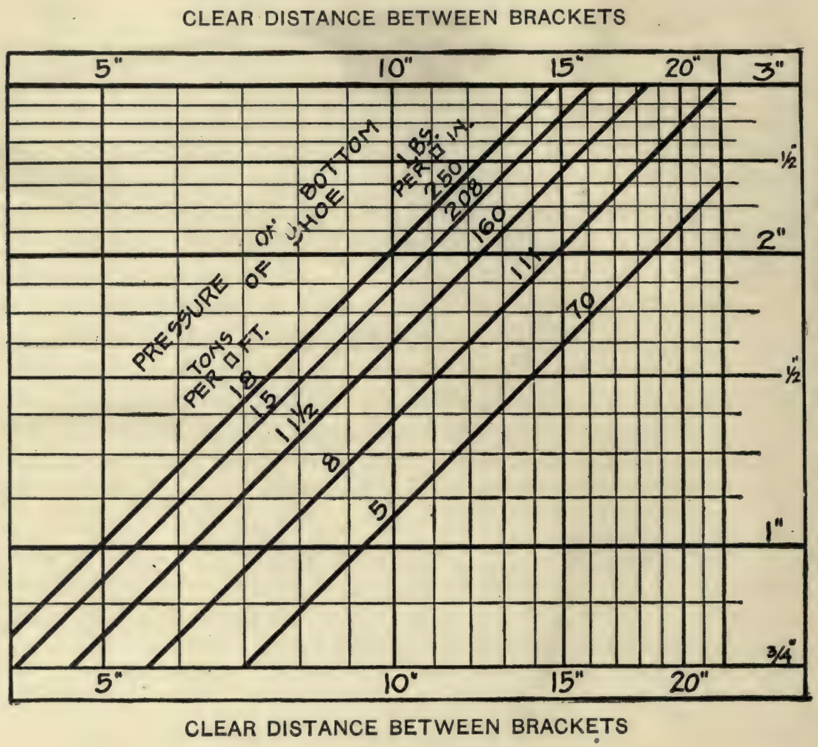
$$A = \frac{B\,L^2}{115\,h} + \frac{W\,a\,b}{83\,L\,h}, \qquad (28)$$

where A = area of cross section of lintel at the centre in square inches.
B = thickness of brick wall in inches.
L = span of lintel in feet.
h = height of cross section at centre of lintel in inches.
W = concentrated load (at distance a and b from each end) in lbs.
(Note:—If more than one concentrated load occurs W a b = W′ a′ b′ + W″ a″ b″ + etc.

*This area includes only the area of the ribs at a distance "a" from the edge of the shoe plus the area of the base plate.

Diagram No. 42

For giving the minimum thickness for bottom plate of cast-iron shoes.



The width required for a lintel is always known and in the case of brick walls is usually 8, 12, 16, 20, 24 or 28 ins., according to the thickness of the wall above it; with the above formula, by assuming a height for the lintel, say 6, 8, 10 or 12 ins., and deciding upon the use of one or two stems, the area of the cross section is found from the first factor of the above formula if no concentrated load occurs; and from the summation of the factors when one or more concentrated loads occur along with the brick wall load.

CHAPTER XVI. WOODEN BEAMS AND POSTS.

For structural purposes wood is almost exclusively employed in rectangular form. This uniformity of section makes the application of wood to framing a comparatively simple problem; and lends itself peculiarly to independent diagram treatment for the solution of such problems. This will be evident from the description of the diagrams presented in this chapter without further discussion on the mechanics of the subject, the treatment of which has been fully covered in Part I.

SAFE LOADS ON WOODEN JOISTS.

Two sets of diagrams are given, one set (Diagrams Nos. 43 and 44) for the strength of white pine, spruce or chestnut joists, for various depths from 3 to 16 ins.; the other set (Diagrams Nos. 45 and 46) for yellow pine and locust.

DIAGRAMS FOR WOODEN JOISTS:—In each diagram abscissas represent span of joist in feet; the ordinates represent spacing of joists in inches; curves on the diagrams show safe load* in pounds per square foot of floor. The spans represented vary from 3 to 30 ft., and the spacings from 12 to 24 ins., and the load per square foot from 1 to 1,000 lbs.

The load lines show a bend which indicates where deflection enters as a factor. For spans to the right of the bend the beams are designed for a limiting deflection of one four-hundredth of the span.

NOTE:—If oak joists are to be used, work out the problem by each of the preceding sets of diagrams, and take a mean of the two results.

It will be evident that floor planking 3 ins. or over in thickness can also be figured by these diagrams.

SAFE LOADS ON WOODEN GIRDERS.

Two sets of diagrams (including Nos. 47 to 50) are also given for these as in the preceding.

DIAGRAMS FOR WOODEN GIRDERS:—These diagrams are constructed the same as those for joists. The depths of girders represented by the diagrams run from 6 to 16 ins.; spans from 3

*It is to be remembered that the diagrams are for beams 1 in. wide; for a beam 2 ins. wide double the load obtained from the diagram; for a beam 3 ins. wide, triple the load, etc.

to 25 ft. are shown, and loads from 1 to 100 lbs. per sq. ft. The spacing of girders represented by the ordinates is given in feet and runs from 7 to 18 ft. The limiting deflection is same as in the preceding diagrams. The diagrams are also for girders 1 in. wide.

SAFE LOADS ON WOODEN POSTS.

The strength of wooden posts is represented on Diagram No. 51. The general arrangement of this diagram is similar to those for steel and cast iron columns. It is constructed on the basis of the stresses provided by the "Code" (N. Y. C.) and is a combination of three distinct diagrams representing, respectively, white pine, white oak, and yellow pine. In all three diagrams abscissas represent column ratios, ordinates represent area of section, and the curves represent concentric loads.

It will be noted that the upper scale of abscissas differs from the scale at the bottom of the diagram. The former represents the ratio of slenderness of the posts as hitherto defined—the quotient of the length divided by the radius of gyration of the cross section —and is given so that Diagrams Nos. 30, 32 and 34 can be combined with it for the purpose of designing wooden posts for eccentric loads. The latter, the scale at the bottom, represents another ratio much used for wooden posts: the length divided by the least width of the section. This latter ratio is the one commonly used.

Two supplementary ordinate scales are given at the right of the diagram. The first of these shows the usual sizes of posts, at proper vertical intervals to represent the areas given on the ordinate scale on the extreme left. One column is given for square posts, another for round posts.

The other supplementary scales show the weight per lineal foot for any cross section area of post. One column is given for white pine and another for yellow pine and white oak which woods are approximately twice as heavy as white pine.

EXAMPLE:—Posts of white pine, white oak or yellow pine 10" × 10" will carry concentric loads of 32 to 18 tons, 36 to 20 tons, or 40 to 23 tons, respectively, for unsupported lengths of 25 ft. to 8 ft. 4 ins.

Diagram No. 43

For giving the safe load on white pine, spruce or chestnut joists for each inch in breadth.

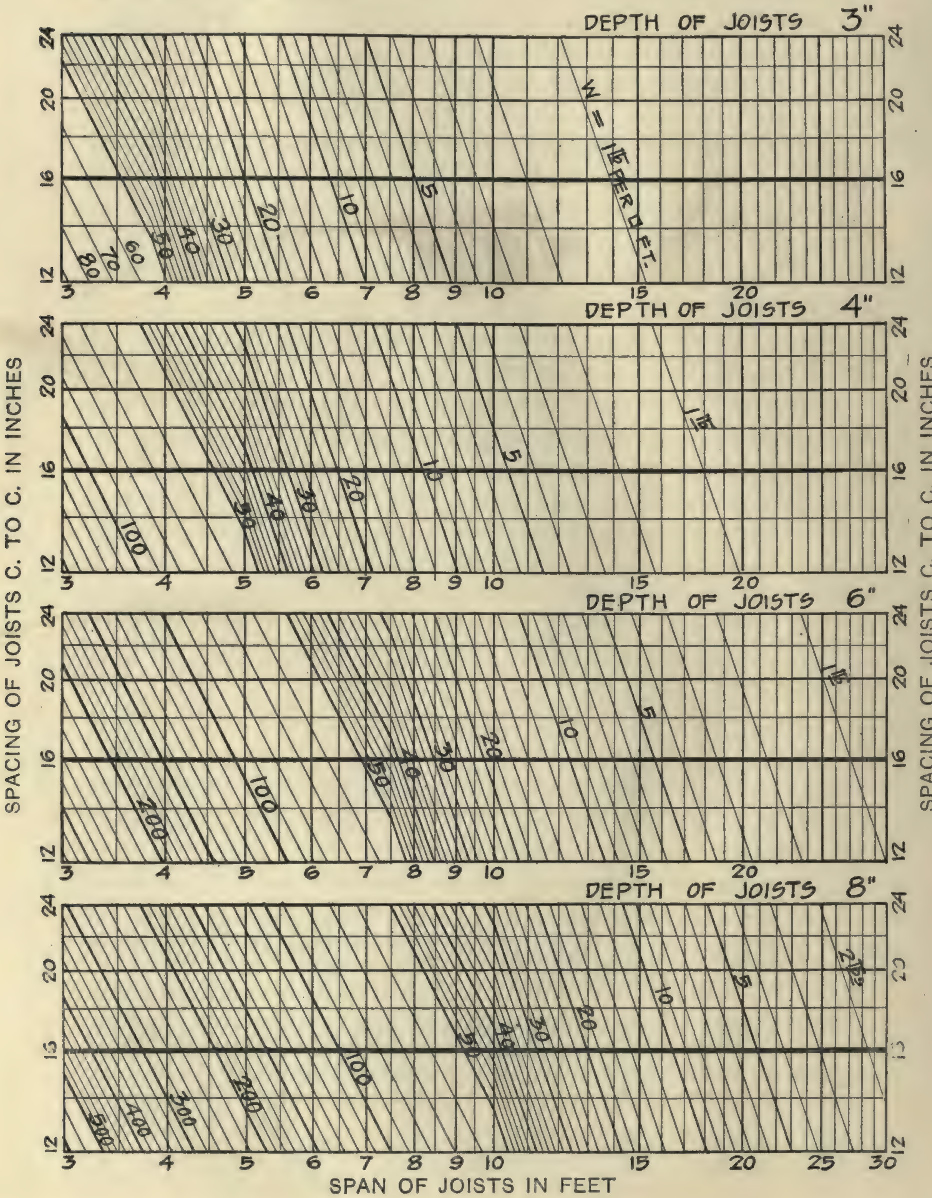


Diagram No. 44

For giving the safe load on white pine, spruce or chestnut joists for each inch in breadth.

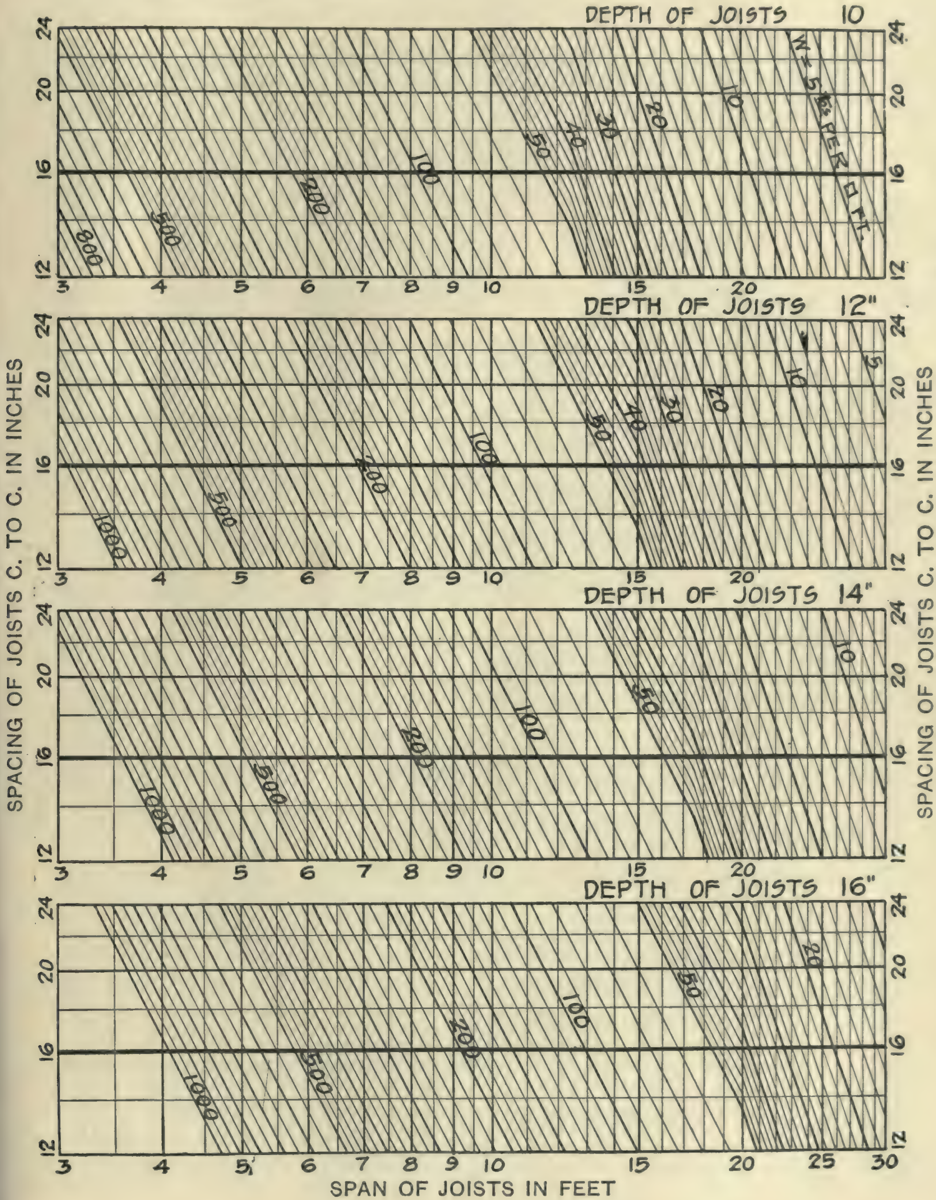


Diagram No. 45

For giving the safe load on long leaf yellow pine or locust joists for each inch in breadth.

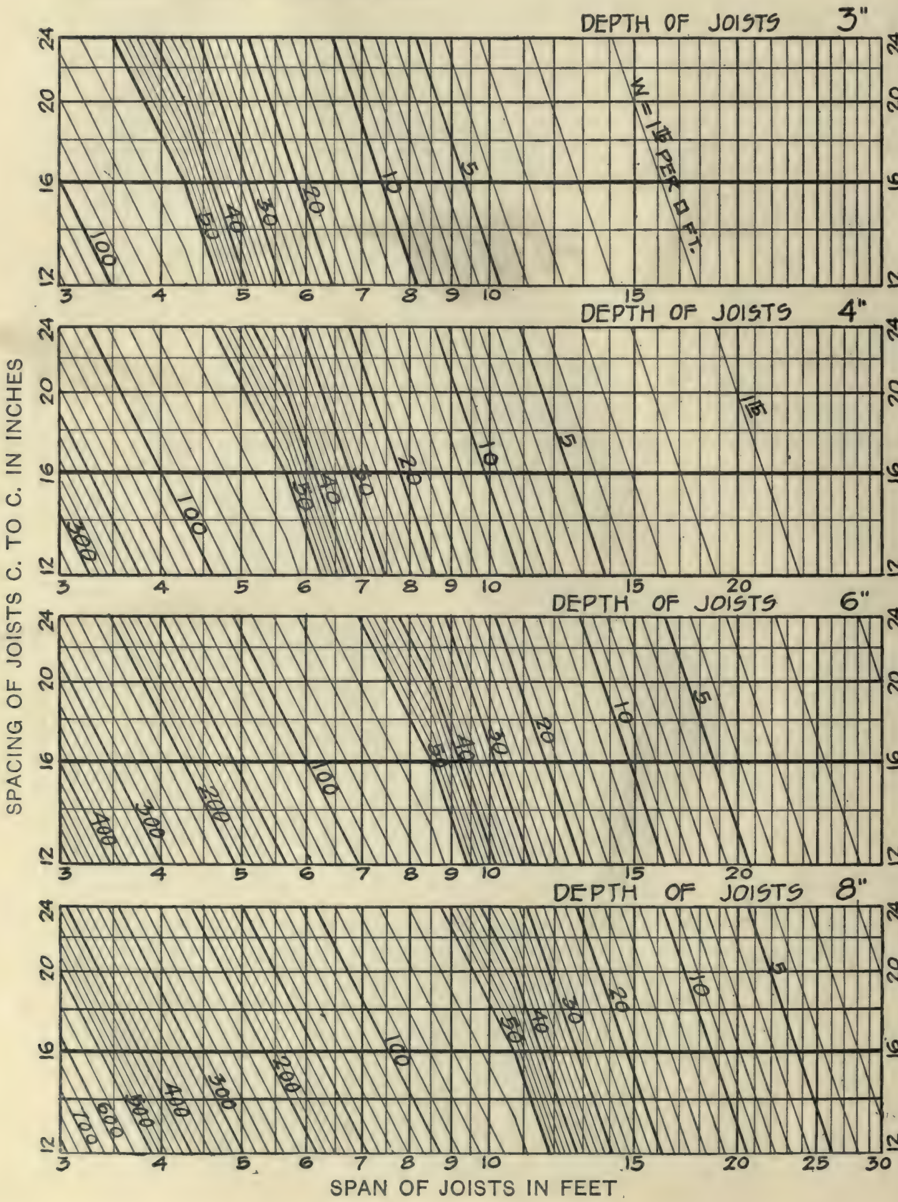


Diagram No. 46

For giving the safe load on long leaf yellow pine or locust joists for each inch in breadth.

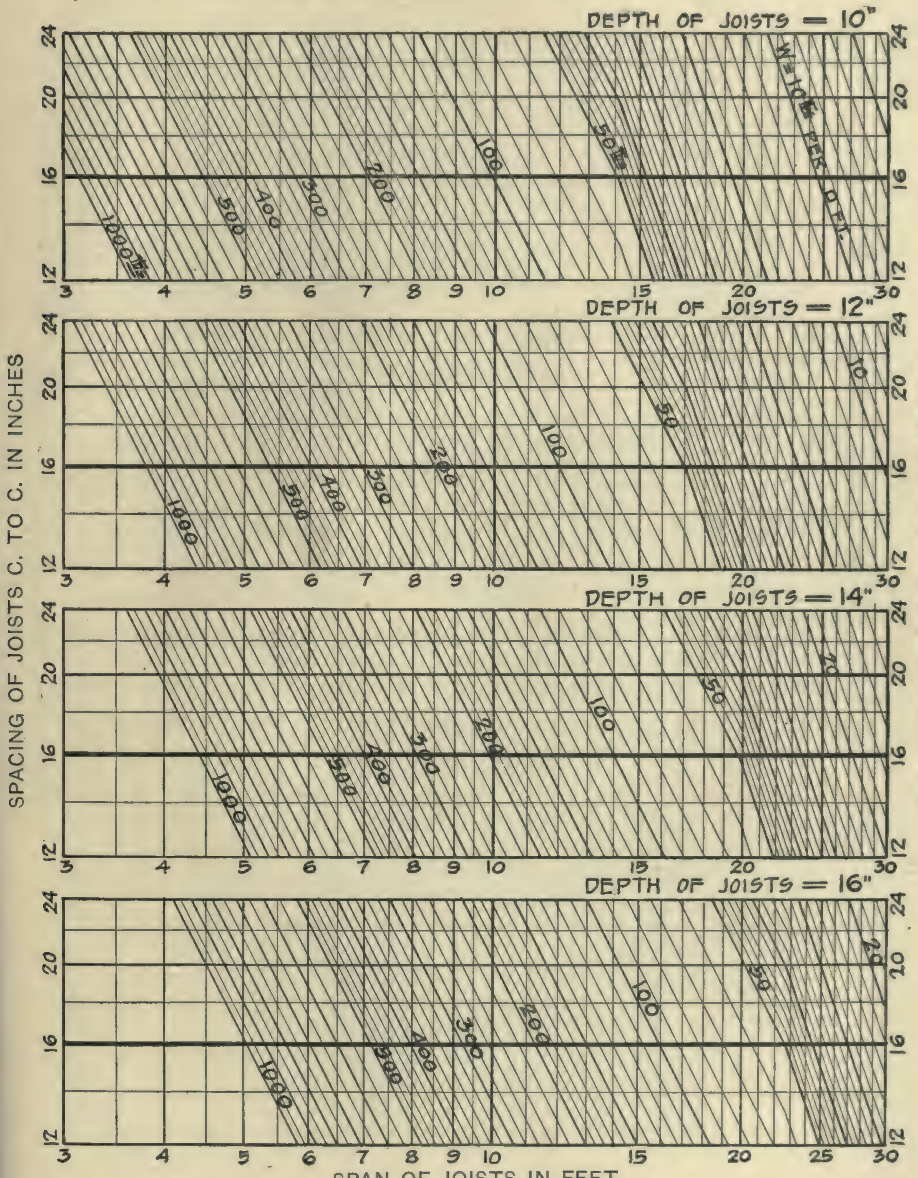


Diagram No. 47

For giving the safe load on white pine, spruce or chestnut girders for each inch in breadth.

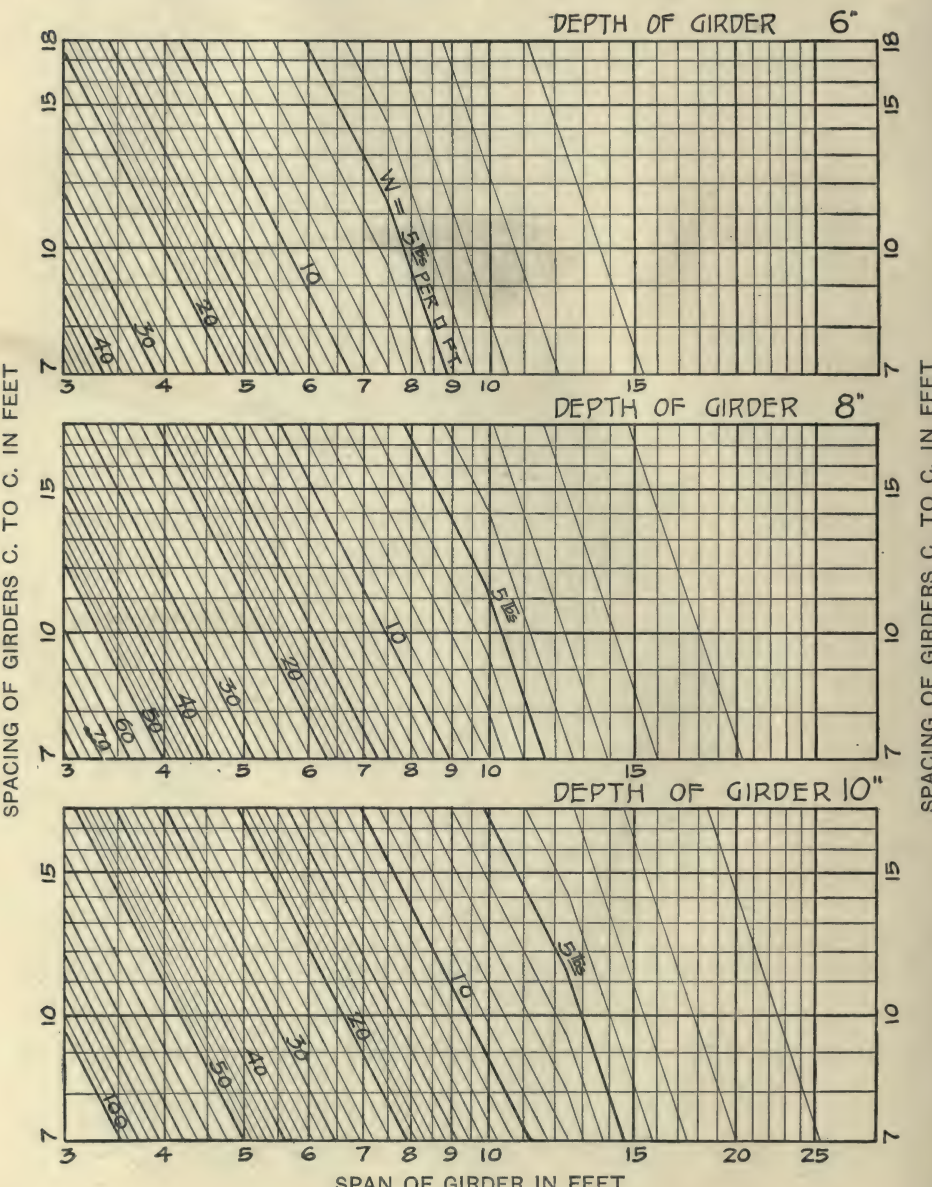


Diagram No. 48

For giving the safe load on white pine, spruce or chestnut girders for each inch in breadth.

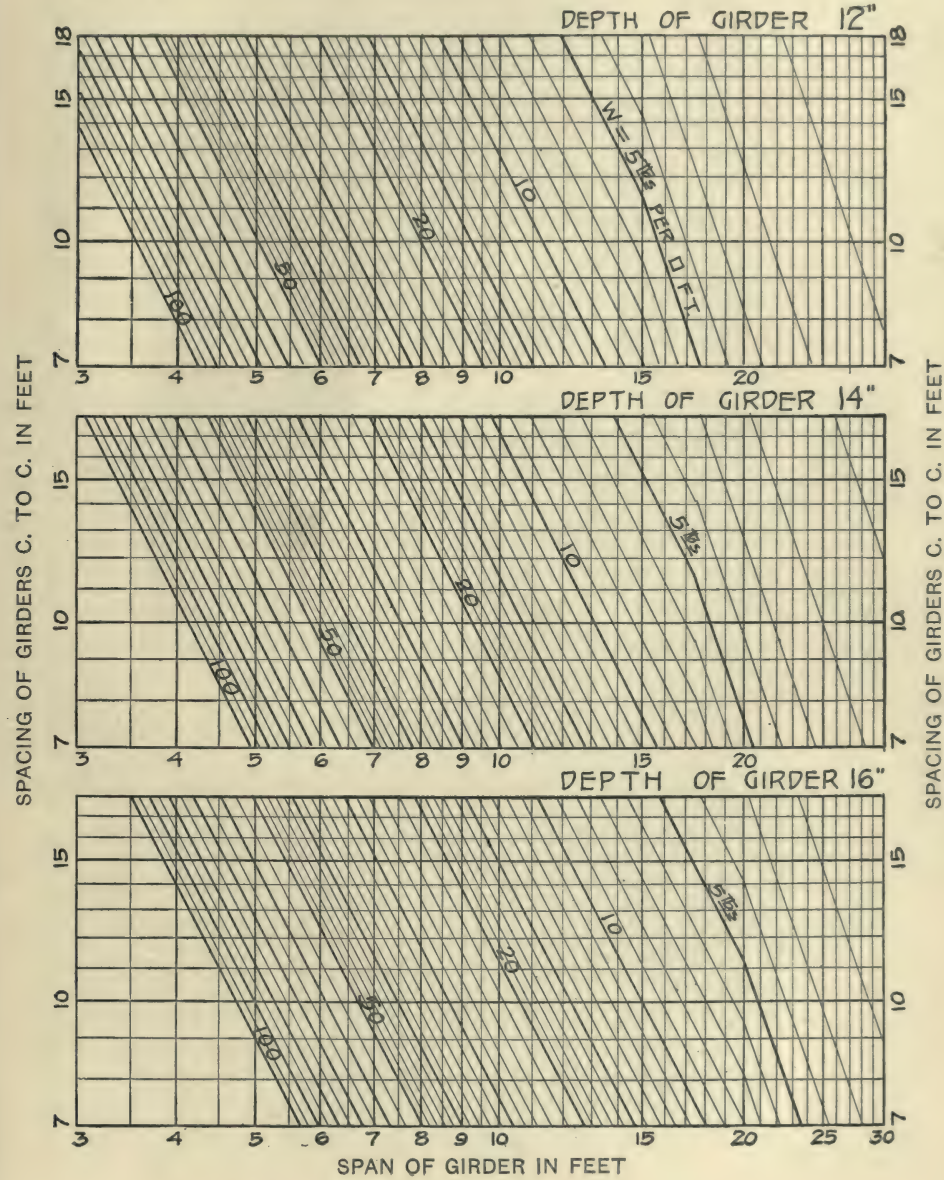


Diagram No. 49

For giving the safe load on long leaf yellow pine or locust girders for each inch in breadth.

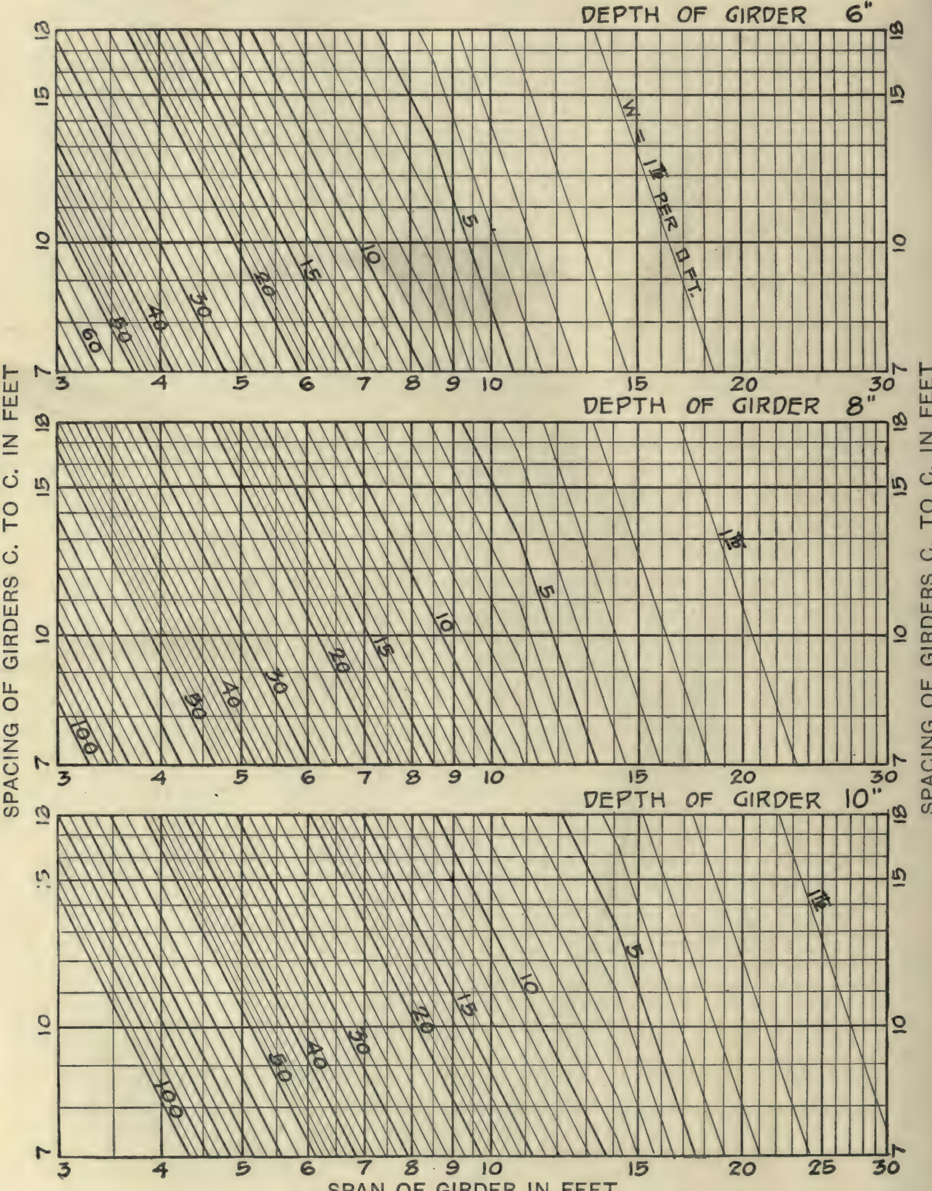


Diagram No. 50

For giving the safe load on long leaf yellow pine or locust girders for each inch in breadth.

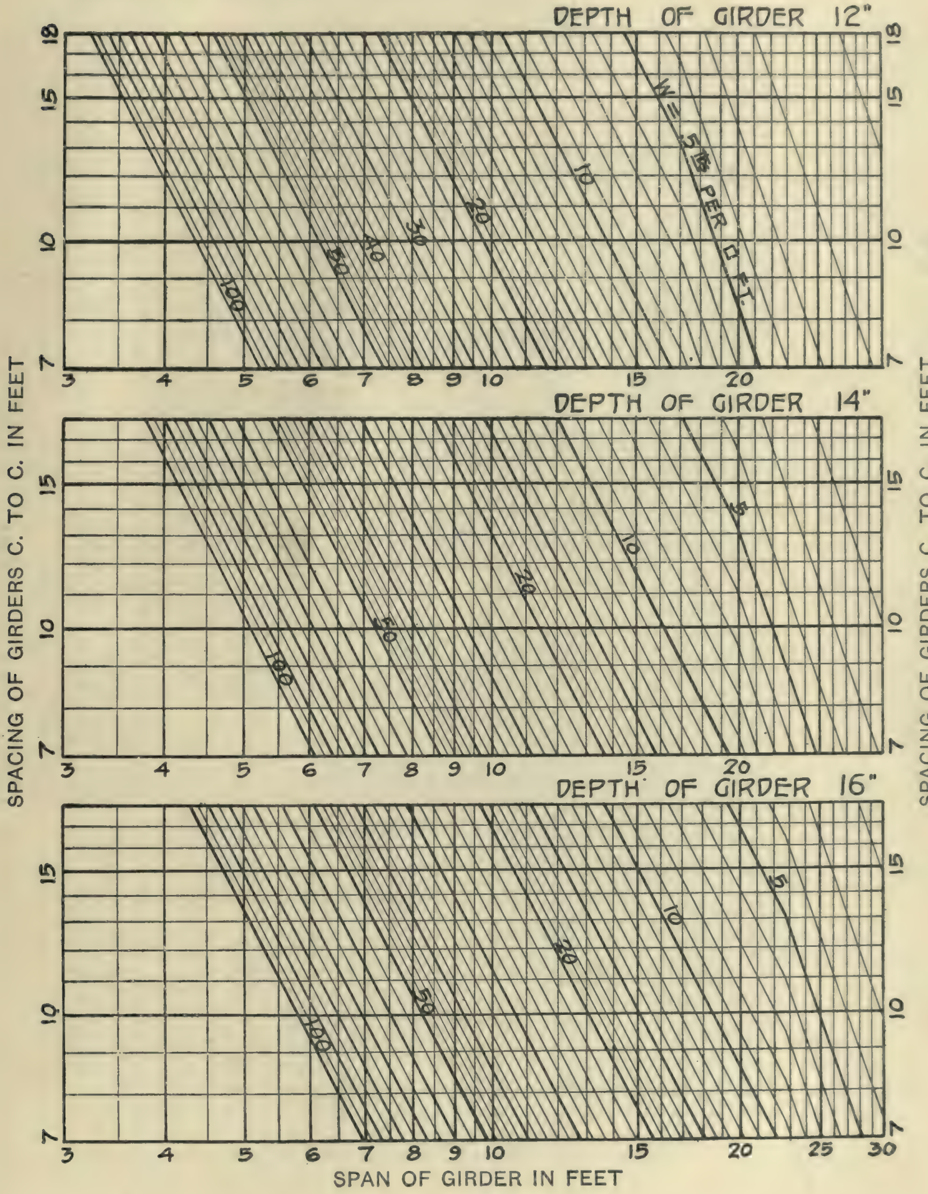
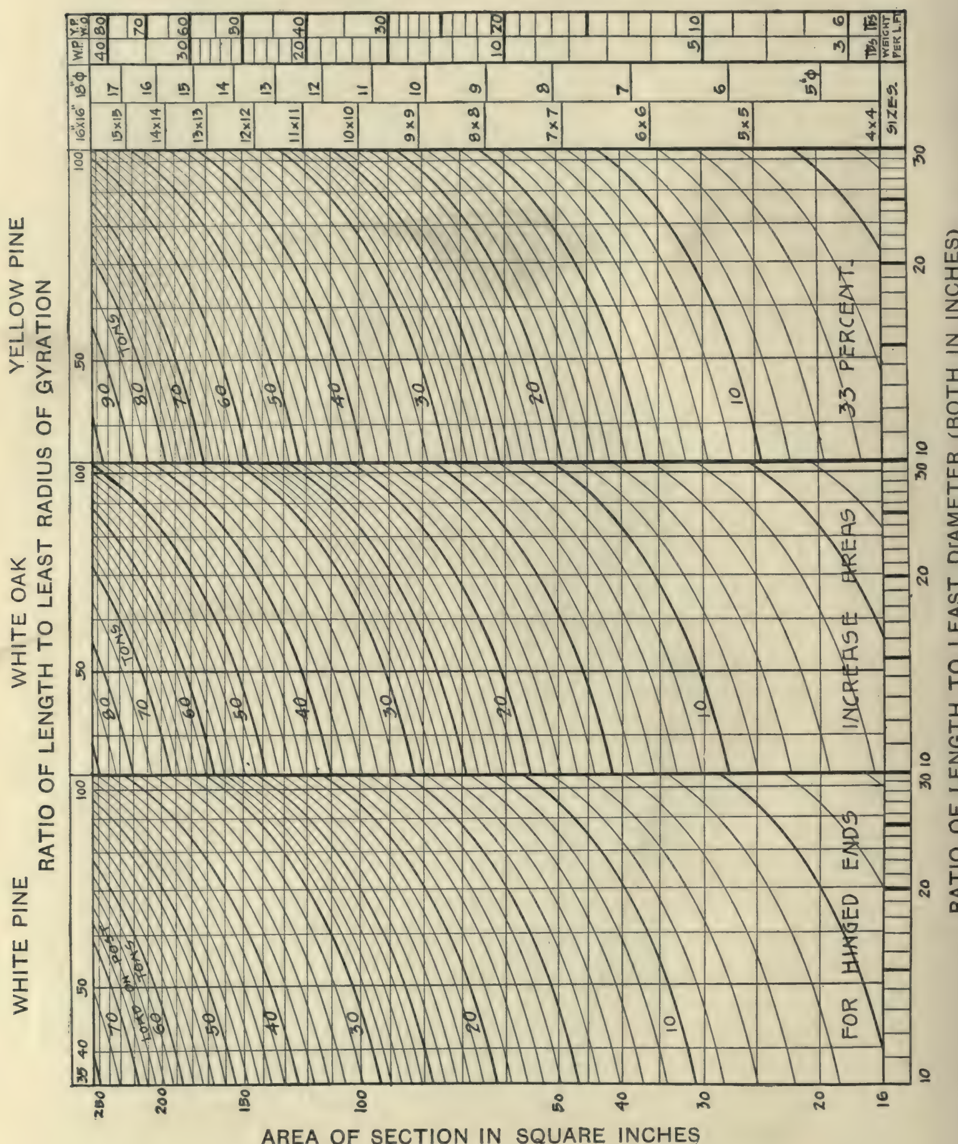


Diagram No. 51

For giving the safe load on wooden posts.



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